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Evaluation of the Empirical Deck Design for Vehicular Bridges

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EVALUATION OF THE EMPIRICAL DECK DESIGN FOR VEHICULAR BRIDGES

by

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Abstract

This research evaluated the feasibility of the empirical design method for reinforced concrete bridge decks for the Florida Department of Transportation [FDOT]. There are currently three methods used for deck design: empirical method, traditional method and finite element method. This research investigated and compared the steel reinforcement ratios and the stress developed in the reinforcing steel for the three different methods of deck design. This study included analysis of 15 bridge models that met the FDOT standards. The main beams were designed and load rated using commercial software to obtain live load deflections. The bridges were checked to verify that they met the empirical method conditions based on the FDOT *Structures Design Guidelines* – January 2009. The reinforced concrete decks were designed using the traditional design method. Then the bridges were analyzed using three-dimensional linear finite element models with moving live loads. The reinforced concrete decks were designed using dead load moment, live load moment, and future wearing surface moment obtained from the finite element models. The required reinforcing steel ratio obtained from the finite element method was compared to the required reinforcing steel ratio obtained from traditional design method and the empirical design method. Based on the type of beams, deck thicknesses, method of analysis, and other assumptions used in this study, in most cases the required reinforcing steel obtained from the finite element design is closer to that obtained from the empirical design method than that obtained from the traditional design method. It is recommended that the reinforcing steel ratio obtained from the empirical design method be used with increased deck thicknesses to control cracking in the bridge decks interior bays.

Keywords: empirical deck design, traditional deck design, deck cracking

Introduction

Background

Most of the new bridge decks in the United States are constructed using reinforced concrete. Based on the *Load and Resistance Factor Design* (LRFD) *Bridge Design Specifications* (American Association of State Highway and Transportation Officials [AASHTO] 2012), Section 9.6.1, bridge decks are allowed to be analyzed using the following three methods:

- Elastic method, also known as traditional method or equivalent strip method, where the deck is divided into strips and analyzed as a reinforced concrete flexural element.
- Empirical method, also known as the Ontario method, if the deck meets certain criteria then the minimum amount of transverse reinforcing steel shall be 0.27 in^2 per foot in the bottom layer and 0.18 in^2 per foot in the top layer. This corresponds to a reinforcing steel ratio of 0.34% in the bottom layer and 0.23% in the top layer, assuming an 8 inch thick deck and using No.5 rebars. However the Florida Department of Transportation [FDOT] *Structures Manual* – January 2009, *Structures Design Guidelines* (SDG) Section 4.2.4 requires the use of no. 5 bars at 12 inches in both directions in the top and the bottom layers.
- Refined method, or finite element modeling, where the deck is modeled using detailed three-dimensional shells or plate elements.

It is assumed that the empirical method provides less reinforcing steel ratio than the elastic method. The lower reinforcing ratio could cause transverse cracking in bridge decks. However, researchers from the Michigan and New York transportation departments have investigated the adequacy of the empirical method and have recommended using it in all situations where the deck falls within LRFD's empirical method guidelines. On the other hand, Barth and Frosch (2001), and Frosch, Blackburn and Radabaugh (2003) maintained that a reinforcing steel ratio of 0.63% obtained from the LRFD traditional method is still necessary for adequate crack control.

Objective

The objective of this research is to verify the feasibility of the empirical design method by using a parametric study that analyzes 15 bridge models. Fifteen models were established using three different span lengths of 70, 80, and 90 feet, with varying beam spacing of 6, 8, 10, 12, and 14 feet. In order to verify the empirical method, all 15 models were analyzed with *Structural Analysis and Design* (STAAD.Pro V8i) software to obtain the dead load moments, future wearing surface moments, and live load moments. The decks were then designed as reinforced concrete flexural element. The reinforcing steel ratio obtained from the finite element analysis is compared to that obtained from the empirical design and the traditional design methods. In addition, the cracking moment in the deck at service, the tensile stress in the deck concrete at service, and the tensile stress in the reinforcing steel at service did not exceed the allowable limits.

Tasks

To accomplish the stated objective, several design and modeling tasks had to be completed:

- All 15 bridge models were designed using commercial software, *SmartBridge*, to obtain the beams' live load deflections, prestressing strand patterns, shear reinforcement, and rating factors.
- The 15 bridge models were checked to verify whether the bridges meet the empirical method requirements based on the FDOT SDG – January 2009.
- The reinforced concrete decks were designed using the traditional method based on AASHTO LRFD.
- The 15 bridge models were analyzed using three-dimensional linear finite element models that include all elements of the structure such as traffic railings, deck, beams, and substructure.
- The decks were designed using dead load, live load, and future wearing surface moments obtained from the finite element models.
- The required reinforcing steel ratio (ρ) obtained from the finite element method was compared to required ρ obtained from the empirical method and to the required ρ obtained from the traditional method to make recommendations whether the empirical method would be acceptable to provide better deck designs with minimal cracking.

Literature Review and Data Collection

Survey

As part of this research project, deck design requirements from the departments of transportation [DOT] in the United States [U.S.] were reviewed. Some of the States' DOTs still use the AASHTO Standard Specifications for bridge deck design and other states' DOTs do not have any bridge design manuals on their websites. The specifications are summarized as follows:

- The Alabama DOT [ALDOT] Bridge Bureau *Structures Design and Detailing Manual* – January 1, 2008 provides a table that shows the required deck thickness and reinforcing steel based on girder type and girder spacing. The table was furnished by the State Bridge Engineer and any exceptions will require his prior approval. The table shows a deck thickness that varies from 7" minimum to 7¾" maximum with girder spacing varying from 4.0' to 10.0'. The main transverse reinforcing steel used is No. 5 bar with spacing between 6½" and 4½". This corresponds to a reinforcing steel ratio of 0.68% to 0.98% per foot.

- The Arizona DOT [ADOT] *Bridge Design Guidelines* – July 2011, Section 9.6.1 allows the reinforced concrete deck to be designed following an approximate elastic method which is referenced in the AASHTO LRFD traditional design method. Refined method of analysis or finite element modeling is only allowed for complex bridges with prior approval from ADOT Bridge Group. The moments due to the unfactored live load shall be obtained from AASHTO LRFD Section 4, Appendix A, Table A4-1 with the negative moment values taken at a distance of 0.0" from the centerline of girder.

- The California DOT [Caltrans] *Bridge Design Practice* – October 2011, Chapter 10, Concrete Decks, allows the design of reinforced concrete decks as transverse strips flexure member which is based on the approximate or traditional method of analysis. The refined method of analysis, based on AASHTO 4.6.3, is recommended for more complex decks analysis, i.e. curved decks, which would require a more detailed analysis. The empirical design method, based on AASHTO 9.7.1 is not permitted for now until further durability testing is completed.

- The Colorado DOT [CODOT] *Bridge Design Manual* – June 1989, Section 8 provides a table that shows the minimum deck thickness and reinforcing steel size and spacing based on the effective span length. The deck thickness varies between 8 and 9 inches with a quarter inch increment. The main transverse reinforcing steel is No. 5 bar with spacing between 9 and 5 inches which corresponds to a reinforcement ratio of 0.47% to 0.85%. This table is based on the Load Factor Design.

- The Connecticut DOT [ConnDOT] *Bridge Design Manual* – 2011, Section 8.1.2.2 requires the decks to be designed using the load factor design. ConnDOT also allows the use of the empirical design method based on the AASHTO LRFD Specifications.

- The Delaware DOT [DelDOT] *Bridge Design Manual* – May 2005, Section 5.3.1.2 does not allow using the empirical design method for decks. DelDOT references AASHTO LRFD Section 4.6.2.1, which is the approximate method of analysis for applying wheel loads.

- The Florida DOT [FDOT] *Structures Manual* – January 2013, SDG does not permit the use of the empirical design method due to future widening or phased construction and associated traffic control impacts. The interior portions of the deck shall be designed using the traditional design method. For the deck overhang and median barriers, the SDG provides a table that shows the required minimum area of reinforcing steel based on the type of traffic railing barrier used on a particular bridge.

- The Georgia DOT [GDOT] *Bridge and Structures Design Manual* – October 2005 uses the service load design for bridge decks to provide a stiffer deck that is subject to less cracking. GDOT provides a deck chart showing the bar size and spacing using the BRSLAB07 design program, and it also assumes the deck is continuous over 3 or more supports with a continuity factor of 0.8 and a minimum deck thickness of 7 inches. The deck overhang is also preferably designed using the service load design, however the load factor design is also allowed since the overhang loading does not occur daily.

- The Idaho Transportation Department [ITD] *LRFD Bridge Design Manual* – November 2005, Section 9.7.2 allows the use of the empirical design method for bridge decks and provides a design aid for determining the deck thickness based on the type of beam used. The concrete girders' types are AASHTO Type 2, AASHTO Type 3, AASHTO Type 4, and Modified Bulb Tee. The steel girders have varying top flange width of 12", 15", 18", and 24". ITD also noted that the empirical design method cannot be used for the Modified Bulb Tee concrete girders when the girder spacing is 5.0' and 5.5'.

- The Illinois DOT [IDOT] *Bridge Manual Design Guides* – April 2012, bridge deck design Section 3.2.1 is based on the traditional method. IDOT Bridge Manual provides a step by step process with a solved example to determine the spacing for No. 5 bars in the top and bottom mats. The standard deck thickness has been increased to 8.0" for beam spacing between 5'-6" and 9'-6".

- The Indiana DOT [INDOT] *Design Manual* – 2012 Chapter 404 Bridge Decks allows the use of the approximate method of analysis, commonly referred to as the equivalent strip method or traditional method, in accordance with AASHTO LRFD 4.6.2. The INDOT Design Manual does not mention whether the empirical deck design method is allowed.

- The Iowa DOT [IDOT] *LRFD Bridge Design Manual* – December 2012, recommends using the strip method for deck design based on AASHTO LRFD 4.6.2.1. The empirical method is to be used only with permission of the Bridge Engineer.

- The Kansas DOT [KDOT] *LRFD Bridge Design Manual* – May 2010, Section 3.9.4 allows the use of the traditional design method for bridge decks and does not use the empirical method. The traditional deck design is based on 8.5" thick deck that includes a 0.5" wearing thickness, and 15 pound per square foot allowance for future wearing surface.

- The Louisiana Department of Transportation and Development [La DOTD] *LRFD Bridge Design Manual* – September 2008, allows the use of both the empirical design method and the traditional design method. The empirical method is not allowed for the overhang design.

The LaDOTD LRFD Bridge Design Manual also lists special provisions related to the concrete material, curing method, and deck thickness when using the empirical deck design.

- The Maine DOT [MDOT] *Bridge Design Guide* – August 2003, Chapter 6, provides 2 tables that show the minimum deck thickness, reinforcing steel size, and spacing based on the maximum girder spacing. The deck thickness varies from 7” to 11” with a half inch increment. The main transverse reinforcing steel is No. 5 bar with a 6-inch spacing which corresponds to a reinforcement ratio of 0.58% to 0.91%. The MDOT *Bridge Design Guide* does not specify what method the design is based on.

- The Massachusetts DOT [massDOT] *LRFD Bridge Manual* – October 2009, Part II, provides design tables showing the required steel reinforcement and deck thickness. Section 3.5.2 of Part I requires using the use of the traditional approximate method of analysis when the beam spacing is outside the table limits. The empirical deck design is not allowed.

- The Michigan DOT [MDOT] *Design Manual – Bridge Design* – Chapter 7 LRFD Section 7.02.19 allows the use of the empirical design method according to AASHTO LRFD 9.7.2.

- The Minnesota DOT [MnDOT] *LRFD Bridge Design Manual* – 2013, Section 9.2.1 requires the use of the traditional approximate method of analysis only. The empirical deck design method shall not be used.

- The Missouri DOT [MoDOT] *Category 751 LRFD Bridge Design Guidelines* Section 751.10.1.4 recommends the use of the equivalent strip method for deck design. The MoDOT LRFD Bridge Design Guidelines also mentions that there are other methods of analysis allowed, such as finite element method, but does not mention the empirical method. The slab portion between girders shall be 8.5" thick for both cast-in-place (CIP) and precast concrete decks.

- The Montana DOT [MDT] *Structures Manual* – August 2002, Chapter 15 provides a figure that shows the slab thickness and reinforcing steel based on the beam spacing. This table is based on the strip method of the AASHTO *Standard Specifications for Highway Bridge.* Rigorous application of the strip method generally results in slightly greater reinforcing steel ration than presented in the figure. Based upon acceptable past performance and the fact that the empirical method of the AASHTO LRFD Specifications requires less reinforcing steel than the strip method, designs in accordance with the figure are considered satisfactory.

- The Nebraska Department of Roads [NDOR] *Bridge Office Policies & Procedures (BOPP) Manual* – 2013 Section 3.1.1 requires the deck be designed using the empirical design method in accordance with current AASHTO LRFD Bridge Design Specifications. The NDOR BOPP Manual also provides the required deck thickness based on the effective span. The top mat shall have No. 4 bars at 12" in both directions while the bottom mat shall have No. 5 bars at 12" in both directions.

- The Nevada DOT [NDOT] *Structures Manual* – September 2008, Chapter 16 allows the use of the traditional approximate method of analysis only. NDOT design practice is to use a 40-kip axle instead of the 32-kip axle specified in the LRFD Specifications. Therefore, the bridge engineer must multiply the design moments shown in LRFD Table A4-1 by 1.25. The empirical method is not allowed by NDOT.

- The New Jersey DOT [NJDOT] *Design Manual for Bridges and Structures* (DMBS)*, 5th Edition* – 2009 Section 3.2 allows the use of the empirical design if the bridge structure entails straight longitudinal superstructure members, otherwise the provisions of the AASHTO LRFD 9.7.3 - Traditional Design shall primarily be followed for concrete deck slab designs. The NJDOT DMBS Section 20.5 – Deck Slab Design and Construction Detailing, also provides a table for deck reinforcing steel design. The table has a beam spacing of 4.25' to 13.17', a deck thickness of 8.25" to 10.75" and the required main top and bottom rebars, the longitudinal top and bottom rebars, and the additional main rebars in the overhang. The table is based on a concrete compressive strength of 4,000 psi, reinforcing yield strength of 60 ksi with 2.5" top cover and a 1.0" bottom cover.

- The New Mexico DOT [NMDOT] *Bridge Procedures and Design Guide* – April 2013 uses the 1979 Bridge Design and Detailing Instructions that show standard details of deck reinforcing steel in a figure. The main reinforcing steel used is No. 5 at 6" for the top and bottom mats. The deck thickness varies with the beam spacing, from 8" for 6'-7" beam spacing to 11" for 11'-10" beam spacing. NMDOT used thinner decks in the past, but practice has shown that thinner decks do not have the long-term durability of the standard decks. Therefore the standard deck should always be used unless approval to use a thinner deck is obtained from the State Bridge Engineer.

- The New York State DOT [NYSDOT] *Bridge Manual* – May 2011, 4th Edition, Section 5.1.5.1 allows the use of the empirical design method for isotropic decks that meet the following conditions:
    - There must be four or more girders in the final cross section of the bridge. A stage construction condition with three girders is permissible; however, the temporary overhangs must be reinforced traditionally.
    - The maximum center-to-center spacing of the girders is 11' and the minimum spacing is 5'.
    - Design slab thickness shall be a minimum of 8" and the total standard deck thickness shall be a minimum of 9½". An 8½" thick deck may be used with solid stainless steel and stainless steel clad reinforcement.
    - The deck is fully cast-in-place and water cured. Only permanent corrugated metal and removable wooden forms shall be permitted (prestressed concrete form units are not allowed).
    - The supporting components are made of spread steel or concrete I-girders.
    - The deck shall be fully composite in both positive and negative moment regions. In negative moment regions, composite section property computations shall only include the area of the longitudinal steel.
    - Isotropic reinforcement may be used with spread concrete box beams provided the reinforcement is adequate to resist flexure for the clear span between beam units.

- The minimum overhang, measured from the centerline of the fascia girder to the fascia, is 2’-6”. If a concrete barrier composite with the deck is used, the minimum overhang is 2’-0”.
- Skew angles up to 45°. Note: For skews above 30° isotropic reinforcement becomes very congested at the end of the slab. Traditional deck slab reinforcement is recommended for skews greater than 30°.

- The North Carolina DOT [NCDOT] *Structures Design Manual* – September 2013, Section 6.2.2 provides standard deck design tables for detailing decks to carry a HL-93 live load. The tables show the deck thickness and reinforcement based on beam spacing. The tables are based on Grade 60 reinforcing steel and a concrete compressive strength of 4,000 psi. The design deck depth is the deck depth less ¼" monolithic wearing surface. NCDOT does not allow the concrete decks to be designed with the empirical method.

- The North Dakota DOT [NDDOT] *LRFD Bridge Design Specifications* – 2004 uses the traditional approximate method of analysis for deck design. The empirical deck design method shall not be used. The deck shall be treated as a continuous beam. Moments as provided in Table A4-1 of the AASHTO LRFD are to be applied at the design section. The use of Table A4-1 must be within the assumptions and limitations listed at the beginning of the appendix.

- The Ohio DOT [ODOT] *Bridge Design Manual* (BDM) – April 2012, Section 300 requires the deck to be designed with the approximate elastic method of analysis in accordance with AAHTO LRFD also known as the traditional design method. The refined method of analysis and the empirical design method, LRFD 9.7.2 are prohibited. The ODOT BDM also provides a concrete deck design aid table that shows the deck thickness, the deck overhang thickness, the transverse steel, and the longitudinal steel bar size and spacing for the top and bottom mats based on an effective span length. It also shows the required additional bars for the deck overhang based on the ODOT approved bridge railings. The design aid table applies only for decks designed in accordance with AASHTO LRFD Bridge Design Specifications and ODOT BDM, and several other design assumptions listed under the table.

- The Oregon DOT [ODOT] *Bridge Design and Drafting Manual* (BDDM) – 2004, Section 1.1.20 does not allow the use of the empirical design method for deck reinforcing steel. The ODOT BDDM explains that excessive deck cracking, apparently due to under reinforcement, precludes the use of this method until further notice.

- The Pennsylvania DOT [PennDOT] *Design Manual Part 4 Structures* – May 2012, Section 9.6.1 requires the concrete decks to be designed in accordance with the approximate elastic method. The refined method and the empirical method are only allowed if approved by the PennDOT Chief Bridge Engineer.

- The Rhode Island DOT [RIDOT] *LRFD Bridge Design Manual* – 2007, Section 9.5 uses the approximate elastic method of analysis for design of concrete decks. The refined method of analysis shall be used only when approved by the Managing Bridge Engineer.

The empirical method of analysis will be considered by the Managing Bridge Engineer on a case-by-case basis.

- The South Carolina DOT [SCDOT] *Bridge Design Manual* – 2006, Section 17.2 allows the use of the strip method only. The use of the empirical deck design is prohibited.

- The Texas DOT [TxDOT] *Bridge Design Manual – LRFD* – (BDM) March 2013 does not use the empirical design method specified in LRFD Article 9.7.2. The TxDOT BDM uses the Traditional Design method specified in LRFD Article 9.7.3.

- The Agency of Transportation in Vermont [VTrans] *Structures Design Manual* – 2010 does not mention whether the deck is designed using a specific method. Section 9.1 provides tables that show the deck thickness and reinforcement based on beam spacing. These tables are based on 4,000 psi concrete and 60,000 psi rebars.

- The Virginia DOT [VDOT] *Structures and Bridge Manuals* (SBM) – December 2012 Volume V Part 2, does not allow the use of the empirical design method. The VDOT SBM also provides a table that shows the required deck thickness, reinforcing steel area and spacing for steel beams and prestressed concrete beams based on the beam spacing.

- The Washington State DOT [WSDOT] *Bridge Design Manual LRFD* – July 2011, Section 5.7 requires that the deck be designed using the traditional design of AASHTO LRFD 9.7.3 with a few modifications. The minimum deck thickness (including wearing surface) is 7.5” for concrete beam bridges, 8.5” for concrete beam bridges that have stay-in-place (SIP) forms and 8.0” for steel beam bridges.

- The West Virginia DOT [WVDOT] *Bridge Design Manual* – 2006 Section 3.2.1 allows the use of the empirical design method provided all required design conditions are met based on AASHTO 9.7.2.4. WVDOT restricts the deck thickness to a minimum of 8.0” for monolithic bridge decks and 8.5” for bridge decks made from specialized concrete overlay. WVDOT has also prepared design drawings for the deck overhangs that meet the dimensional criteria set in these drawings and carry the standard WVDOT barriers. Otherwise the deck overhang shall be design for all loads including the impact loads based on LRFD 9.7.1.5 and the requirements of LRFD 9.7.2.

- The Wisconsin DOT [WisDOT] *LRFD Bridge Manual* – July 2013, allows the use of the empirical design method with prior approval from WisDOT.

- The Wyoming DOT [WYDOT] *Bridge Design Manual* (BDM) – December 2012, Chapter 2 uses the traditional design based on AASHTO LRDF 9.7.3. The WYDOT BDM also provides deck reinforcing steel table that shows the deck thickness, the girder spacing, the bar size, positive, and negative moments based on 12 inch-spacing for transverse bars and maximum longitudinal bar spacing. The design is based on the HL93 Design Loading.

Table 2.1 below summarizes the deck design method by state. Figure 2.1 shows that only 18% or 9 states allow the use of the empirical design on their bridges.

Table 2.1: Bridge Deck Design Method by State

State	Method	State	Method
Alabama	N/A	Montana	N/A
Alaska	N/A	Nebraska	Empirical
Arizona	Traditional	Nevada	Traditional
Arkansas	N/A	New Hampshire	N/A
California	Traditional	New Jersey	Empirical
Colorado	N/A	New Mexico	N/A
Connecticut	Empirical	New York	Empirical
Delaware	Traditional	North Carolina	N/A
Florida	Traditional	North Dakota	Traditional
Georgia	N/A	Ohio	Traditional
Hawaii	N/A	Oklahoma	N/A
Idaho	Empirical	Oregon	Traditional
Illinois	Traditional	Pennsylvania	Traditional
Indiana	Traditional	Rhode Island	Traditional
Iowa	Traditional	South Carolina	Traditional
Kansas	Traditional	South Dakota	N/A
Kentucky	N/A	Tennessee	N/A
Louisiana	Empirical	Texas	Traditional
Maine	N/A	Utah	N/A
Maryland	N/A	Vermont	N/A
Massachusetts	Traditional	Virginia	Traditional
Michigan	Empirical	Washington	Traditional
Minnesota	Traditional	West Virginia	Empirical
Mississippi	N/A	Wisconsin	Empirical
Missouri	Traditional	Wyoming	Traditional

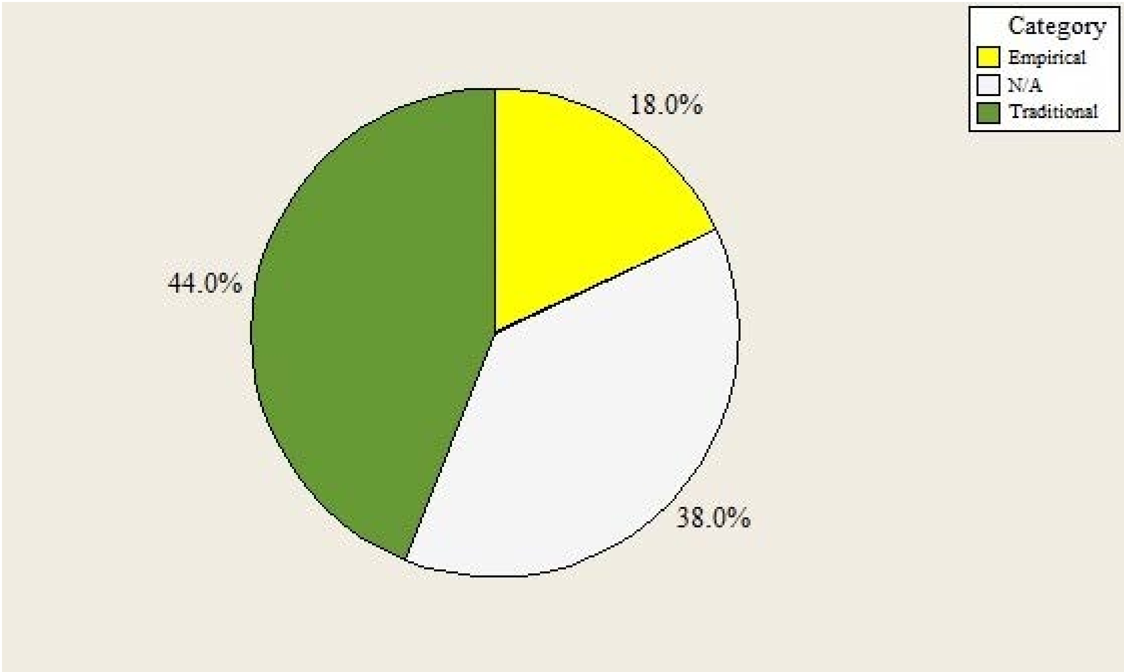


Figure 2.1: Bridge Deck Design Method by State

Literature Review

Traditionally, reinforced concrete bridge decks have been designed using a one foot distribution width with live load moments equations provided as a function of a design truck wheel load based on the AASHTO *Standard Specifications for Highway Bridges*, 2002. This method defines an assumed one foot wide section to carry the vehicular load negative and positive bending moments. The deck is divided into strips perpendicular to the supporting beams/girders (LRFD Section 4.6.2.1, 2012). The deck interior section, between the fascia girders, is assumed to be continuous over the supporting beams/girders. Primary flexural reinforcement, in the transverse direction, is then selected based on traditional procedures for the design of one-way reinforced concrete flexural slabs. The deck design is also checked in the secondary direction for flexure, torsion, and resistance as a percentage of the primary direction. Special procedures are used for the design of deck overhangs that include vehicular live-load and dynamic impact loading of the parapet. The specifications also detail the placing of additional reinforcement in the negative bending regions of continuous spans.

Several analytical and experimental studies have been conducted to verify the empirical design method (also known as the Ontario method). Some of these studies have led to the conclusion that the traditional method provides high amount of reinforcing steel (Fang, Worley, Burns & Klinger, 1986; Tsui, Burns & Klinger, 1986; Fang, Worley, Burns & Klinger, 1990). Investigation into the performance of concrete decks revealed that the primary structural action by which these slabs resist concentrated wheel loads is not flexure but a complex internal membrane stress state known as internal arching. This action takes place when concrete cracks in the positive moment region and the neutral axis shifting upward. The action is resisted by in-plane membrane forces that develop as a result of lateral restraints provided by the adjacent concrete deck, girders, and other parts of the bridge acting compositely with the deck.

An experimental and analytical study conducted by Fang, et al., (1986), for the Texas State Department of Highways and Public Transportation on two types of concrete decks: cast-in-place and precast decks, showed that the results predicted by the analytical models correlated with the experimental findings. Fang, et al., (1986) tested a full-scale bridge deck (cast-in-place and precast) on steel girders, that was designed in accordance with the Ontario method (also known as the empirical method) and having only about 60 percent of the reinforcing steel required by AASHTO strip method. The test specimen consisted of 7.5" thick concrete deck made composite with three W36x150 steel beams. The beams were 49' long c/c of the bearings and spaced at 7' with 3.25' overhang. A three-dimensional thick shell element, as shown in Figure 2.2, was used to model the concrete deck with edge restraints using *Structural Analysis Program* (SAP IV) software and the compared to the beam-theory solution. Then the composite action of the deck slab and girder was modeled using a thick shell element and three-dimensional beam elements combination, as shown in Figure 2.3.

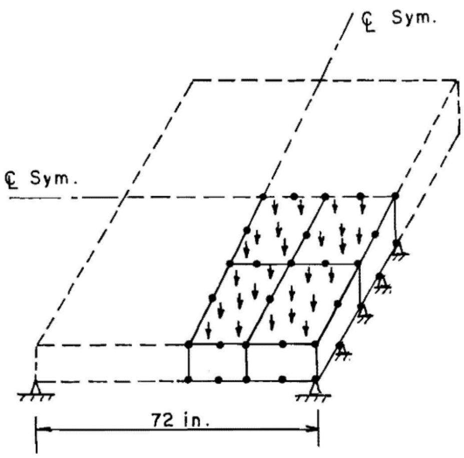
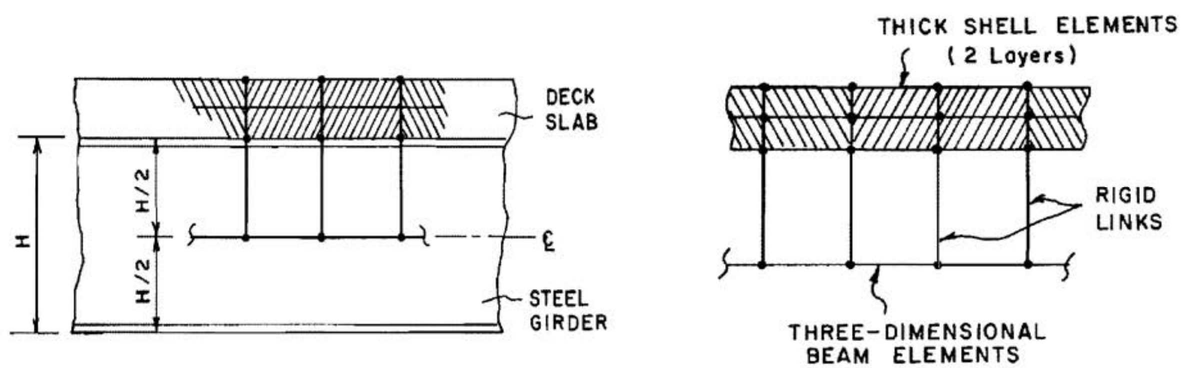


Figure 2.2: Model of restrained thick shell elements.

Adapted from "Behavior of Ontario-Type Bridge Decks on Steel Girders," by Fang, et al., 1986, p. 38.



Actual composite elements

Finite element model

Figure 2.3: Finite element model of composite girder.

Adapted from "Behavior of Ontario-Type Bridge Decks on Steel Girders," by Fang, et al., 1986, p. 47.

The analysis was based on a smeared cracking model with a sequential linear approach, the cracked concrete was assumed to remain continuous and the cracks were smeared as shown in Figure 2.4. The test procedure consisted of loading the specimen with a Highway Standard (HS20) truck with a maximum wheel load of 16 kips magnified by the maximum impact factor. This resulted in maximum wheel service load of 20.8 kips. Fatigue tests used a maximum wheel load of 26 kips, which is 25% higher than the service load. The load was to four locations on the deck simultaneously with wheel lines at 6 feet transversely and 20 feet longitudinally.

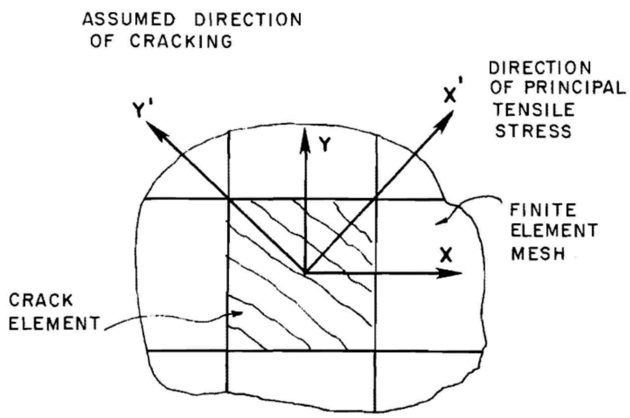


Figure 2.4: Schematic of smeared crack model.

Adapted from “Behavior of Ontario-Type Bridge Decks on Steel Girders,” by Fang, et al., 1986, p. 52.

The test showed that the deck performed satisfactorily under the AASHTO deign loads, the behavior of the deck was linear under service and overload conditions and wasn’t affected by fatigue loading and compressive membrane forces did not affect the performance of the bridge at loads below cracking.

Another experimental and analytical study accomplished by Tsui, et al., (1986), which was a continuation of the study conducted by Fang, et al., (1986), concentrated on dealing with the negative moment behavior and ultimate capacity of the deck under concentrated loads. For the negative moment test, the bridge was setup to simulate a continuous structure. The test setup was designed to produce the maximum moment in the bridge over both end supports. Comparing the analytical and experimental results showed that the analytical model generally overestimated the deflections of the girders except at the midspan section of the interior girder. But since the deflections were very small, the analytical results were consistent with the experiments. For the concentrated load test, the results showed that the general punching shear model gives the closest prediction tot eh experimental results. The flexural capacities of the deck as assumed by the yield-line theory, with or without arching action, were higher than actual failure loads. The values predicted by the ACI and AASHTO formulas based on punching shear model were lower that the test values. The experimental observations showed that punching shear was the failure mode. In summary, decks designed using the empirical method with less reinforcement than traditional method bridge deck, punching shear was the critical failure mode under concentrated load. The deck flexural capacity predicted using yield-line theory is not likely to govern in a conventionally designed deck. Both ACI and AASHTO formulas gave very conservative estimates of the deck's punching shear capacity. Both the cast-in-place and precast prestressed decks resulted in satisfactory behavior at midspan, under static tandem loads.

A study by Csagoly (1989) conducted in Ontario showed that unreinforced concrete decks could carry up to 80% of the maximum load carried by a comparable reinforced concrete deck. This remarkable performance can be explained by the presence of massive in-plane membrane forces produced by lateral restraints. Confinement is provided by any structure elements attached to the deck or to which the deck is connected to, i.e. beams diaphragms and railing. Punching tests conducted on existing bridges using a 100 kip load, showed a very small deflection of the deck and indicated the safe structural strength of existing decks. Csagoly (1989) also concluded that concrete strength, presence of diaphragms, spacing of shear studs, negative moments, dead load stresses, load position, and previous adjacent failures have little or no effect on the capacity of the decks. Rather the factors Csagoly found to be of significant importance were level of restraint, ratio of span to thickness, and amount of reinforcement. He also recommended standardized prefabricated decks with a four layer isotropic reinforcing mat where a typical mat would have No. 4 bars at 12" on center, as shown in Figure 2.5. The mat would be manufactured in a continuous process to a standard width of 8'-6" to allow for transportation without special permit.

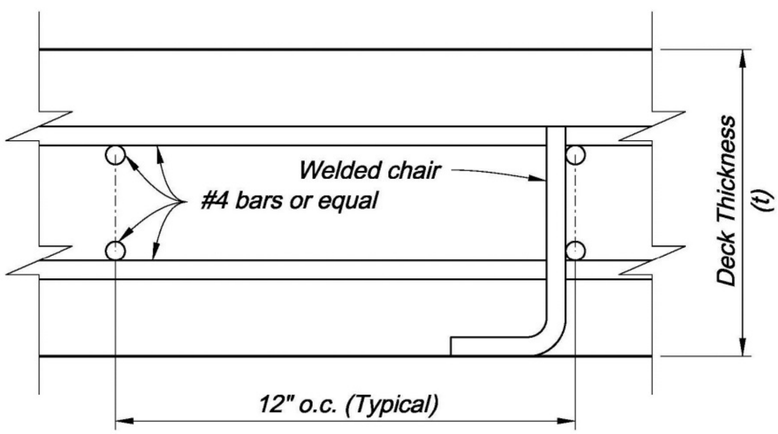


Figure 2.5: Proposed four-layer reinforcement mat.

A Michigan DOT study that was conducted by Nowak, Szerszen and Ferrand (2003) analyzed the design procedure and load rating of isotropic bridge decks. The study evaluated two empirical decks, one supported on steel girders and the other on prestressed concrete girders. Stress distribution was examined at the top and bottom of the decks and through the deck thickness. Stress level due to dead load and live load was found to be less than the cracking limit, and it was concluded that the empirical method provides adequate amount of rebars in the deck. However, restraint shrinkage and the tension stress exceeded the modulus of rupture in the concrete, thus additional reinforcing steel was recommended for isotropic decks supported on deeper steel prestressed concrete girders.

A study conducted by Jansson (2008) compared the performance of ten isotropic in-service decks to several conventional decks on parallel bridges. Isotropic decks are designed using the empirical method where the reinforcing steel is the same size and spacing in both the transverse and longitudinal direction in the top and bottom mats. The conventional decks were based on AASHTO's traditional method of analysis were the decks are assumed to act as flexural beams with a specified strip width. The bridges studied were based on the state of Michigan environment and vehicle loads. This research found that the crack widths and densities are comparable between the two designs, with the isotropic decks showing less transverse cracking and more longitudinal cracking than conventional decks. For both design methods, the cracking was proportional to the beam spacing and truck traffic volume. The study recommendations were to continue using the empirical method, evaluate the cost savings when determining whether to use the isotropic design, special requirements for SIP forms angles placement to minimize cracks and continue to monitor the isotropic deck performance to confirm that long-term serviceability and durability are not reduced in comparison with the traditional deck design.

Barth and Frosch (2001) claimed that a reinforcing steel ratio of 0.63% obtained from the LRFD traditional method is still necessary for adequate crack control. Barth and Frosch (2001) also claimed that control of cracking is as important as the control of deflection in flexural elements. Cracks caused by flexural and tensile stresses reduce the service life of the structure by allowing moisture and oxygen to reach the reinforcing steel and causing corrosion. Crack width prediction equations have been established for beams and thick one-way slabs to calculate cracks width at the bottom of the beam and the reinforcement level, as shown in Equations 1 and 2.

$$w_b = 0.091\sqrt[3]{t_b A}\beta(f_s - 5) \times 10^{-3} \quad (1)$$

$$w_s = (0.091\sqrt[3]{t_b A}/(1 + t_s/h_1))(f_s - 5) \times 10^{-3} \quad (2)$$

where: $w_b =$ most probable maximum crack width at bottom of beam, in.

w_s = most probable maximum crack width at level of reinforcement, in.

$f_s =$ reinforcing steel stress, ksi.

A = area of concrete symmetric with reinforcing steel divided by number of bars, in.^{2}.

$t_b =$ bottom cover to center of bar, in.

$t_s =$ side cover to center of bar, in.

$\beta =$ ratio of distance between neutral axis and tension face to distance between neutral axis and reinforcing steel about 1.20 in beams.

$h_1 =$ distance from neutral axis to the reinforcing steel, in.

Another equation that they used to limit cracking in beams and one-way slab limits the spacing of reinforcing steel closest to the surface in tension (see Equation 3).

$$s(in.) = [(540/f_s - 2.5C_c)] < 12\ (36/f_s)\ or\ 12\ in. \quad (3)$$

where: f_s = calculated stress in reinforcing steel at service load (ksi)

= unfactored moment divided by the product of steel area and internal moment arm, or

= 0.6

C_c = clear cover from the nearest surface in tension to the tension steel, in.

s = spacing to flexural tension reinforcement nearest to the surface if the extreme tension face, in.

Another investigation of bridge deck cracking by Frosch, et al., (2003) determined that transverse deck cracking was caused by restrained shrinkage of the concrete deck, while longitudinal deck cracking was produced by a combination of restrained shrinkage and the use of angles to hold the SIP forms with a leg turned into the deck. Frosch, et al., (2003) recommended that the concrete compressive strength be minimized as to not exacerbate deck cracking, controlling the early-age deck cracking by limiting the maximum bar spacing to 6", increasing the reinforcement amount as recommended in Equation 4 to prevent yielding of the reinforcement that can result in uncontrolled crack development. For 4,000 psi concrete and 60,000 psi reinforcement, Equation 4 would result in 0.63% steel reinforcement ratio in the deck cross-section. They also recommended discontinuing the use of the SIP forms and if the SIP forms are to be used, their angle legs should be turned down.

$$A_s = 6(\sqrt{f'_c}/f_y)\ A_g \tag{4}$$

where: A_g = gross area of section, in^2

A_s = area of reinforcing steel, in^2

f'_c = concrete compressive strength, psi

f_y = yield strength of reinforcing steel, psi

Research by Nielsen, Schmeckpeper, Shiner, and Blanford (2010) for ITD consisted of comparing the ITD design to those used by other state DOTs. Nielsen et al., (2010) recommended reducing the spacing and increasing the size of the reinforcing steel in the deck to alleviate deck cracking issues in ITD's bridge decks. The rebar spacing was recommended to be 6" maximum to reduce shrinkage and drying cracks' widths and the deck thickness 8.5" minimum to increase its stiffness.

Deck Design in Canadian Standards

The *Canadian Highway Bridge Design Code* – November 2006, Clause 8.18.1 allows the use of the empirical design method where decks do not need to be analyzed, except for the negative moment region in the overhang and in the continuous spans over the supports. It also gives the option of using flexural design methods as an alternative to the empirical method. The minimum deck thickness shall be 175 mm (7 inches) and the clear distance between the top and bottom transverse reinforcement shall be a minimum of 55 mm (2.25 inches). In order to use the empirical design method, the deck must meet all of the following conditions:

- The deck thickness between the fascia beams must be uniform.
- The deck is made composite with the supporting beams.
- The supporting beams are parallel to each other and the beams' bearing lines are also parallel.
- The beam spacing to deck thickness ratio is less than 18.0.
- The beam spacing is less than 4.0 meters (13.0 feet).
- The deck extends sufficiently beyond the fascia beams to provide the full development length for the bottom transverse reinforcement.
- The longitudinal reinforcement shall be provided in the deck in the negative moment region for continuous spans.
- The deck shall contain two mats of reinforcing steel near the top and bottom faces, with a minimum reinforcement ratio, ρ, of 0.003 in each direction, as shown in Figure 2.6.
- When the deck is supported on parallel beams, the reinforcement bars closest to the top and bottom faces are placed perpendicular to bearing lines or are placed on a skew parallel to the bearing lines.

- The reinforcement ratio, ρ, may be reduced to 0.002 where the deck with the reduced reinforcement can be satisfactorily constructed and the reduction of ρ below 0.003 is approved.
- Where the transverse reinforcing bars are placed on a skew, the reinforcement ratio for these bars is not less than $\rho/\cos^2\theta$, where θ is the skew angle.
- Where the unsupported length of the edge stiffening beam, S_e, exceeds 5 m (16.5 feet), the reinforcement ratio, ρ, in the exterior regions of the deck slab is increased to 0.006, as shown in Figure 2.7.
- The spacing of the reinforcement in each direction and in each assembly does not exceed 300 mm (12.0 in).

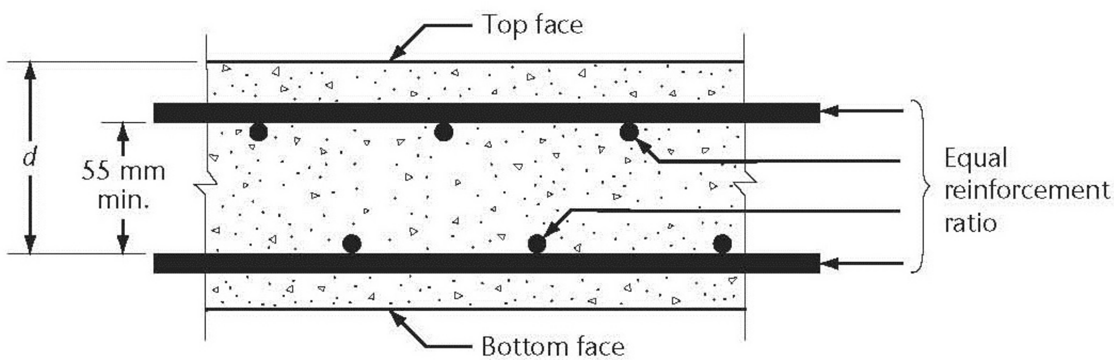


Figure 2.6: Reinforcement in cast-in-place deck.

Adapted from "Canadian Highway Bridge Design Code," by Canadian Standards Association, 2006, p. 368.

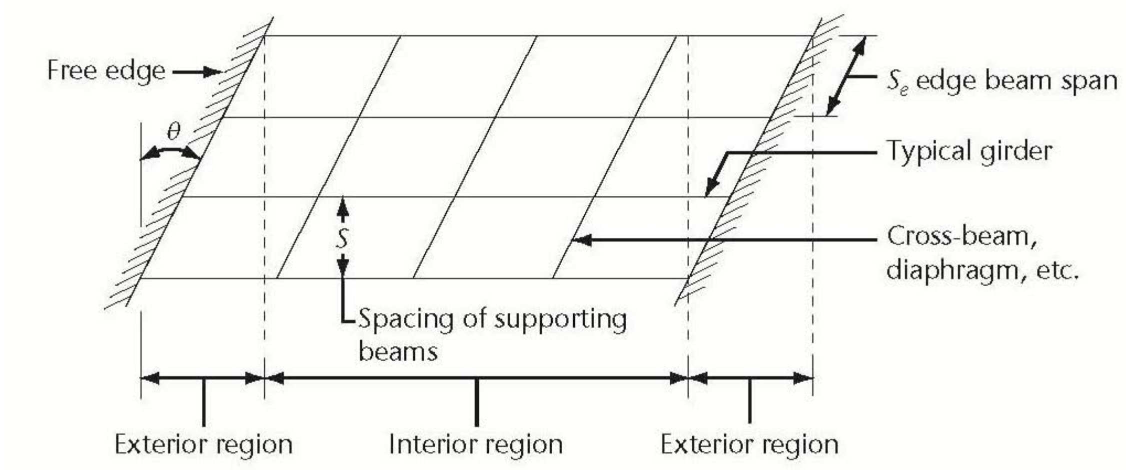


Figure 2.7: Reinforcement for CIP decks designed using the empirical method.

Adapted from “Canadian Highway Bridge Design Code,” by Canadian Standards Association, 2006, p. 369.

Design of Bridge Decks

Empirical Design Method

Arching Action

Arching Action is defined in the AASHTO LRFD as “A structural phenomenon in which wheel loads are transmitted primarily by compressive struts formed in the slab”. In order to use the empirical design for bridge decks, the concrete deck is assumed to resist the concentrated wheel loads through internal membrane stress, also known as internal arching and not through traditional flexural resistance.

The arching action takes place when cracks develop in the positive moment region of the reinforced concrete deck which results in shifting the neutral axis toward the compression zone. The arching action is resisted by in-plane membrane forces that develop as a result of lateral confinement provided by the surrounding concrete deck, rigid accessories, and supporting components acting compositely with the deck.

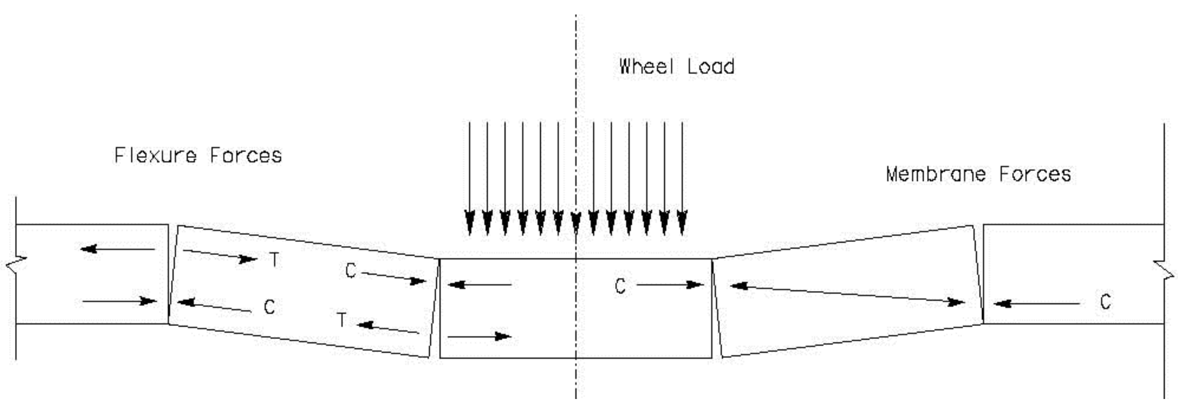


Figure 3.1: Concrete deck showing flexure and membrane forces

Adapted from “Bridge Design Practice,” by Caltrans, 2011 p. 10-2

Conditions to use the empirical method

The empirical design shall only apply to that portion of the deck that is between the fascia girders (LRFD 9.7.2.2). The deck thickness shall not consider the sacrificial thickness due to grinding, grooving, and wear. The empirical design may be used if the following conditions are satisfied:

- Cross-frames or diaphragms are used throughout the cross section at lines of support.
- For cross sections having torsionally stiff girders, i.e. box beams, intermediate diaphragms are provided at a maximum spacing of 25 feet, or the deck is analyzed for transverse bending over the webs and additional reinforcement provided is needed.
- The supporting beams/girders are made of steel and/or concrete.
- The deck is cast-in-place and water cured.
- The deck has a uniform thickness, except at haunches and other local thickening.
- The effective length to design thickness ratio is less than 18.0 and greater than 6.0. This would result in a beam spacing of 4.0 feet minimum and 12.0 feet maximum, based on a typical FDOT deck with thickness of 8.0 inches.
- The core depth of the slab is not less than 4.0 inches as shown in Figure 3.2.
- The minimum thickness of the deck is 7.0 inches, excluding any sacrificial wearing surface. Based on the FDOT SDG, the minimum deck thickness shall be 8.0 inches for “Short Bridges” and 8.5 inches for “Long Bridges”.
- There is a minimum overhang beyond the centerline of the fascia girder of 5.0 times the thickness of the deck; or the overhang is 3.0 times the thickness of the deck and a structurally continuous barrier is made composite with the deck overhang.
- The 28-day compressive strength of the deck concrete is at least 4.0 ksi.

- The deck is made composite with the supporting structural components.

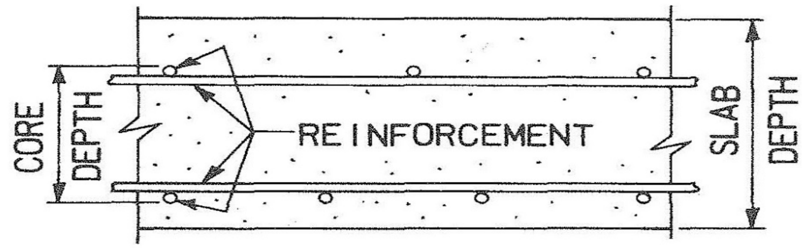


Figure 3.2: Core of a concrete deck.

Adapted from "LRFD Bridge Design Specifications," by AASHTO, 2012 p. 9-11.

Reinforcing Steel Required

In the empirical design method also known as the Ontario method, if the deck meets the criteria listed above, then the minimum amount of transverse reinforcing steel shall be 0.27 in^2 per foot in the bottom layer and 0.18 in^2 per foot in the top layer. This corresponds to a reinforcing steel ratio of 0.34% in the bottom layer and 0.23% in the top layer, assuming an 8-inch thick deck and using No.5 rebars. However, the FDOT SDG Section 4.2.4 states the following: "For Category 1 structures meeting the criteria in LRFD [9.7.2.4] and are not subject to either staged construction or future widening, design deck slabs by the Empirical Design method of LRFD [9.7.2]. In lieu of the minimum area and maximum spacing reinforcing requirements of LRFD [9.7.2.5], use no. 5 bars at 12-inch centers in both directions in both the top and bottom layers."

Traditional Design (The Equivalent Strip Method)

Based on the AASHTO *LRFD Bridge Desi*gn *Specifications* – 2012, the traditional design method shall apply to concrete decks that have four layers of reinforcement, two in each direction. The concrete decks shall have a minimum thickness of 7.0" unless otherwise approved by the owner. The traditional design assumes that the deck is a flexural element. The design loads for the decks consist of dead loads of structural components, i.e. deck self-weight, SIP forms, traffic railing, and sidewalk, dead loads of wearing surfaces and utilities and the vehicular live load. The positive and negative bending moments due to dead loads can be calculated by assuming the deck continuous over three supports. Since in any typical reinforced concrete deck, the slab spans primarily in the transverse direction or perpendicular to the traffic, the live bending moments should be based only on the axles of the AASHTO HL-93 design truck or design tandem (LRFD 3.6.1.3.3). Single wheel loads and design lane load should not be applied. The live load effect may be determined using the approximate method of analysis or the refined methods of analysis, i.e. finite element modeling.

In the approximate method of analysis, the deck is divided into strips perpendicular to the main longitudinal girders. The deck reinforcement is designed for the maximum positive and negative moments in any regions in the deck. The equivalent width of an interior strip of a deck may be obtained from Table 3.1. For decks spanning in a direction parallel to traffic, the strips' widths are limited to 40.0" for open grids and 144.0" for all other decks. For decks spanning in a direction transverse to traffic, the strips' widths have no limits. The deck overhang may be analyzed by replacing the outside row of wheel loads with a uniformly distributed line load of 1.0 kip per linear foot located at 1.0' from the face of the railing.

Table 3.1: Equivalent Strip Widths.

Adapted from "LRFD Bridge Design Specifications," by AASHTO, 2012 p. 4-24.

Type of Deck	Direction of Primary Strip Relative to Traffic	Width of Primary Strip (in.)
Concrete:		
• Cast-in-place	Overhang	$45.0 + 10.0X$
	Either Parallel or Perpendicular	$+M$: $26.0 + 6.6S$ $-M$: $48.0 + 3.0S$
• Cast-in-place with stay-in-place concrete formwork	Either Parallel or Perpendicular	$+M$: $26.0 + 6.6S$ $-M$: $48.0 + 3.0S$
• Precast, post-tensioned	Either Parallel or Perpendicular	$+M$: $26.0 + 6.6S$ $-M$: $48.0 + 3.0S$
Steel:		
• Open grid	Main Bars	$1.25P + 4.0S_b$
• Filled or partially filled grid	Main Bars	Article 4.6.2.1.8 applies
• Unfilled, composite grids	Main Bars	Article 4.6.2.1.8 applies
Wood:		
• Prefabricated glulam		
○ Noninterconnected	Parallel Perpendicular	$2.0h + 30.0$ $2.0h + 40.0$
○ Interconnected	Parallel Perpendicular	$90.0 + 0.84L$ $4.0h + 30.0$
• Stress-laminated	Parallel Perpendicular	$0.8S + 108.0$ $10.0S + 24.0$
• Spike-laminated		
○ Continuous decks or interconnected panels	Parallel Perpendicular	$2.0h + 30.0$ $4.0h + 40.0$
○ Noninterconnected panels	Parallel Perpendicular	$2.0h + 30.0$ $2.0h + 40.0$

Where:

S = spacing of supporting components (ft)
h = depth of deck (in)
L = span length of deck (ft)
X = distance from load to point of support (ft)
P = axle load (kip)

+M = positive moment
-M = negative moment
S_b = spacing of gtid bars (in)

The positive and negative moments in the deck due to the vehicular loads have been calculated by AASHTO and presented in Appendix A4 of the LRFD. This table is reproduced in Table 3.2 below.

Table 3.2: Maximum Live Load Moments per unit width, kip-foot per foot. Adapted from "LRFD Bridge Design Specifications," by AASHTO, 2012 p. 4-98.

S		Positive Moment	Negative Moment						
			Distance from CL of Girder to Design Section for Negative Moment						
			0.0 in.	3 in.	6 in.	9 in.	12 in.	18 in.	24 in.
4'	–0"	4.68	2.68	2.07	1.74	1.60	1.50	1.34	1.25
4'	–3"	4.66	2.73	2.25	1.95	1.74	1.57	1.33	1.20
4'	–6"	4.63	3.00	2.58	2.19	1.90	1.65	1.32	1.18
4'	–9"	4.64	3.38	2.90	2.43	2.07	1.74	1.29	1.20
5'	–0"	4.65	3.74	3.20	2.66	2.24	1.83	1.26	1.12
5'	–3"	4.67	4.06	3.47	2.89	2.41	1.95	1.28	0.98
5'	–6"	4.71	4.36	3.73	3.11	2.58	2.07	1.30	0.99
5'	–9"	4.77	4.63	3.97	3.31	2.73	2.19	1.32	1.02
6'	–0"	4.83	4.88	4.19	3.50	2.88	2.31	1.39	1.07
6'	–3"	4.91	5.10	4.39	3.68	3.02	2.42	1.45	1.13
6'	–6"	5.00	5.31	4.57	3.84	3.15	2.53	1.50	1.20
6'	–9"	5.10	5.50	4.74	3.99	3.27	2.64	1.58	1.28
7'	–0"	5.21	5.98	5.17	4.36	3.56	2.84	1.63	1.37
7'	–3"	5.32	6.13	5.31	4.49	3.68	2.96	1.65	1.51
7'	–6"	5.44	6.26	5.43	4.61	3.78	3.15	1.88	1.72
7'	–9"	5.56	6.38	5.54	4.71	3.88	3.30	2.21	1.94
8'	–0"	5.69	6.48	5.65	4.81	3.98	3.43	2.49	2.16
8'	–3"	5.83	6.58	5.74	4.90	4.06	3.53	2.74	2.37
8'	–6"	5.99	6.66	5.82	4.98	4.14	3.61	2.96	2.58
8'	–9"	6.14	6.74	5.90	5.06	4.22	3.67	3.15	2.79
9'	–0"	6.29	6.81	5.97	5.13	4.28	3.71	3.31	3.00
9'	–3"	6.44	6.87	6.03	5.19	4.40	3.82	3.47	3.20
9'	–6"	6.59	7.15	6.31	5.46	4.66	4.04	3.68	3.39
9'	–9"	6.74	7.51	6.65	5.80	4.94	4.21	3.89	3.58
10'	–0"	6.89	7.85	6.99	6.13	5.26	4.41	4.09	3.77
10'	–3"	7.03	8.19	7.32	6.45	5.58	4.71	4.29	3.96
10'	–6"	7.17	8.52	7.64	6.77	5.89	5.02	4.48	4.15
10'	–9"	7.32	8.83	7.95	7.08	6.20	5.32	4.68	4.34
11'	–0"	7.46	9.14	8.26	7.38	6.50	5.62	4.86	4.52
11'	–3"	7.60	9.44	8.55	7.67	6.79	5.91	5.04	4.70
11'	–6"	7.74	9.72	8.84	7.96	7.07	6.19	5.22	4.87
11'	–9"	7.88	10.01	9.12	8.24	7.36	6.47	5.40	5.05
12'	–0"	8.01	10.28	9.40	8.51	7.63	6.74	5.56	5.21
12'	–3"	8.15	10.55	9.67	8.78	7.90	7.02	5.75	5.38
12'	–6"	8.28	10.81	9.93	9.04	8.16	7.28	5.97	5.54
12'	–9"	8.41	11.06	10.18	9.30	8.42	7.54	6.18	5.70
13'	–0"	8.54	11.31	10.43	9.55	8.67	7.79	6.38	5.86
13'	–3"	8.66	11.55	10.67	9.80	8.92	8.04	6.59	6.01
13'	–6"	8.78	11.79	10.91	10.03	9.16	8.28	6.79	6.16
13'	–9"	8.90	12.02	11.14	10.27	9.40	8.52	6.99	6.30
14'	–0"	9.02	12.24	11.37	10.50	9.63	8.76	7.18	6.45

Finite Element Design

In the finite element design, the flexural and torsional deformation of the deck should be considered and the vertical shear deformation may be neglected (LRFD 4.6.3.2). The deck can be assumed to act as an isotropic plate element where the thickness is uniform and the stiffness is almost equal in all directions. It could be assumed to act as an orthotropic plate element, where the flexural stiffness may be uniformly distributed along the cross-section of the deck and the torsional rigidity is not contributed by a solid plate only. The refined orthotropic deck analysis could also be used where direct wheel loads are applied to the deck structure. Three dimensional shell or solid finite element model could be used for the refined orthotropic deck model utilizing the following simplifying assumptions: linear elastic behavior, plane sections remain plane, small deflection theory, residual stresses, and imperfections are neglected (LRFD 4.6.3.2.4).

Analytical Modeling of Bridge Decks

As stated in the objective, 15 bridge models were analyzed to verify the feasibility of the empirical design method based on AASHTO LRFD and the FDOT SDG – January 2009. The models were first designed following the FDOT SDG requirements for bridge deck thickness of 8" minimum and deck concrete compressive strength. The superstructures were supported on Florida-I 36 beams (FIB36) based on FDOT design standard 20036 and the Instructions for Design Standards (IDS) Index 20010 Series Prestressed Florida-I Beams. Then the deck thicknesses were increased to obtain favorable results for the empirical design. The 15 models were established using three different span lengths of 70, 80, and 90 feet, with varying beam spacing of 6, 8, 10, 12, and 14 feet. The five different typical sections, shown in Figures 4.1 through 4.5, were aimed to accommodate a minimum of 3 design lanes based on AASHTO LRFD 3.6.1.1.1, with a constant overhang of 4' on both sides. The beam ends were directly lined up over 18" prestressed concrete square piles.

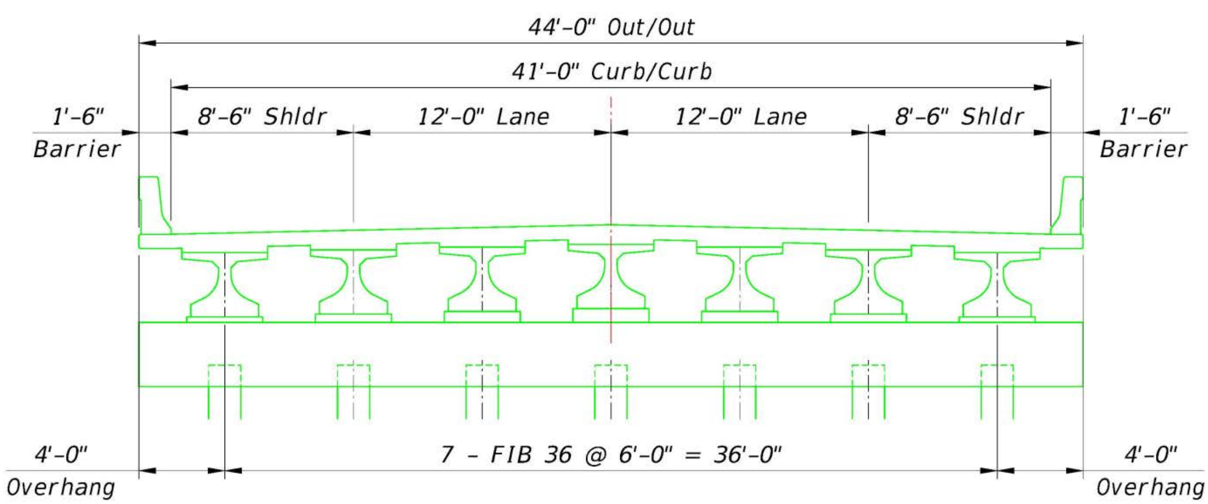


Figure 4.1: Typical section for 6-foot beam spacing

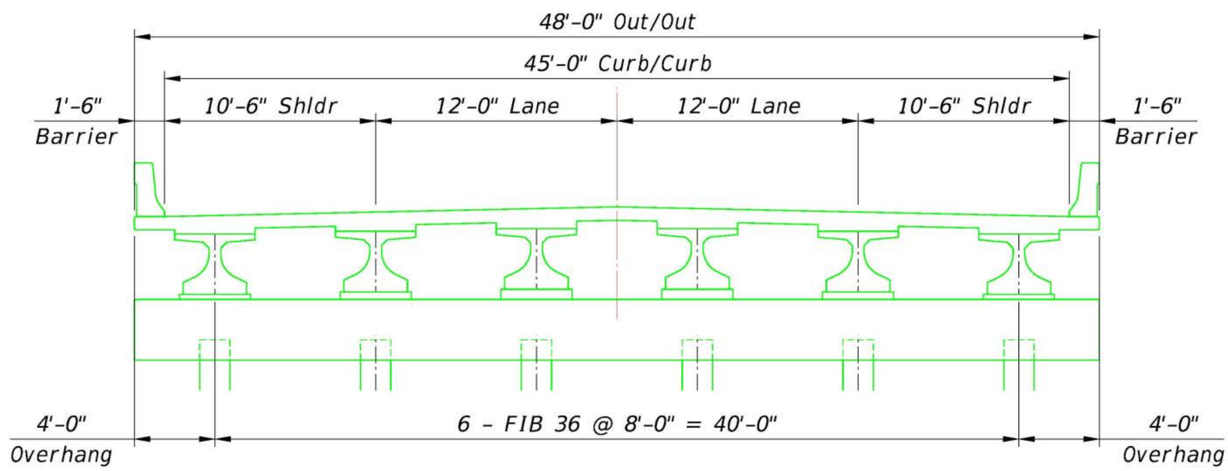


Figure 4.2: Typical section for 8-foot beam spacing

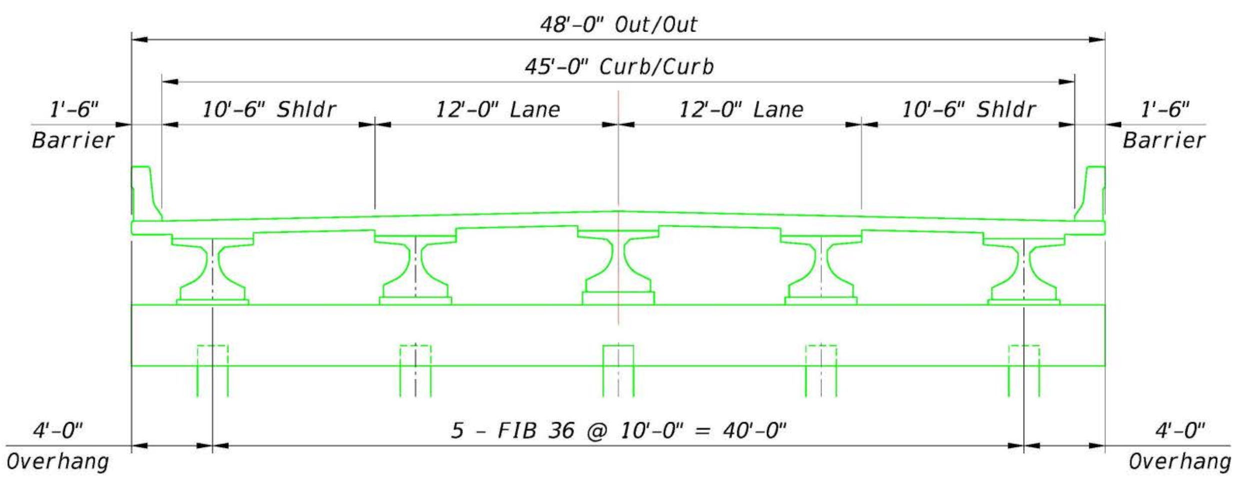


Figure 4.3: Typical section for 10-foot beam spacing

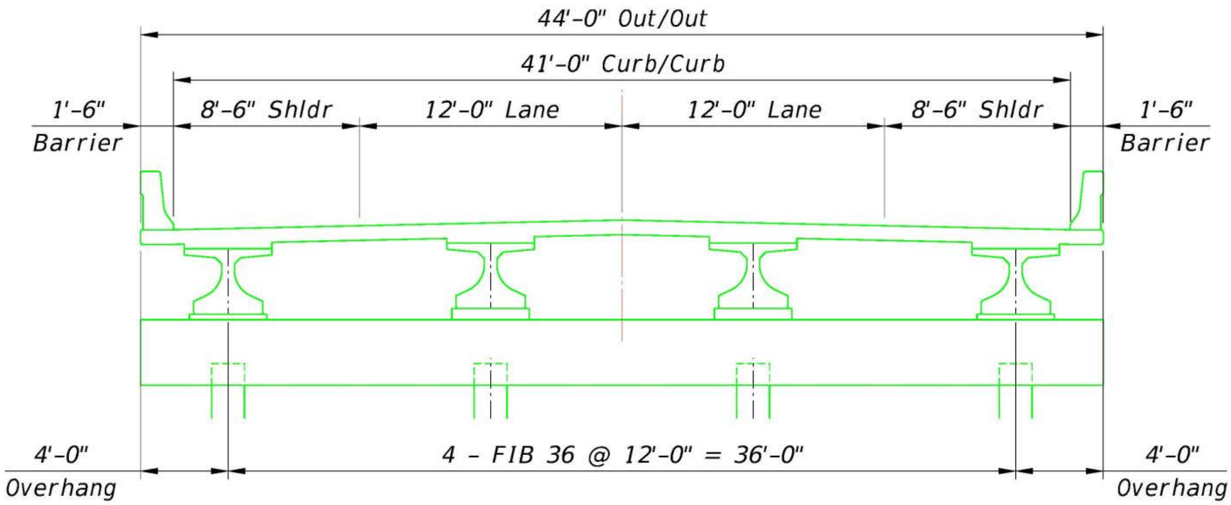


Figure 4.4: Typical section for 12-foot beam spacing

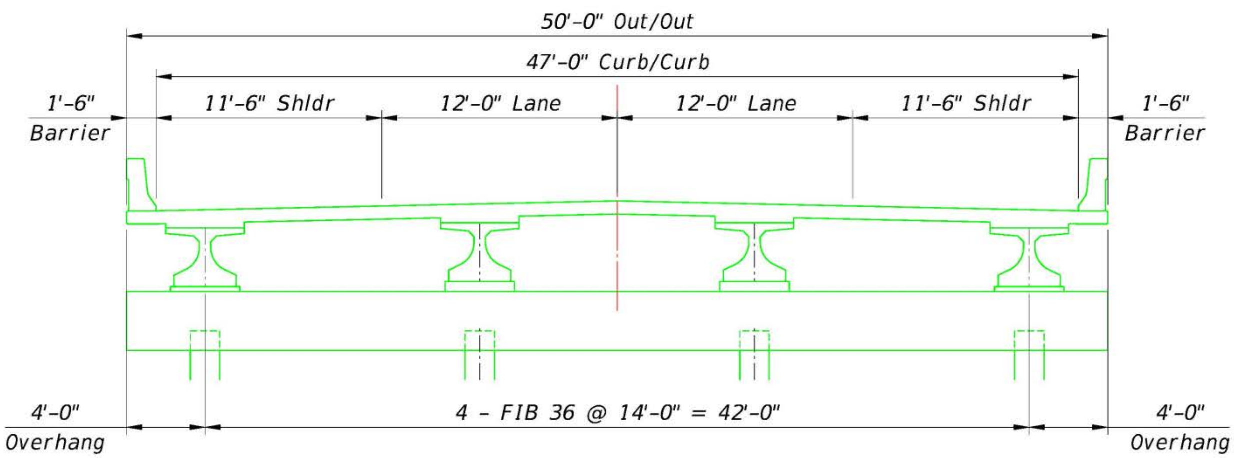


Figure 4.5: Typical section for 14-foot beam spacing

Prestressed FIB36 Beam Design and Load Rating

To obtain the live load deflections due to the AASHTO HL-93 vehicular load, the FIB36 beams were designed with *SmartBridge* to obtain the prestressing strands quantity, de-bonding layout, and the shear reinforcement. The environmental classification was assumed to be Extremely Aggressive based on the FDOT SDG Table 1.4.3-1 which requires a Class IV deck concrete. The concrete strength used was 5,500 psi, based on the FDOT *Standard Specifications for Road and Bridge Construction* (Specs) Section 346-3, for Class IV concrete. The concrete used for the Prestressed FIB36 beams was Class VI with a concrete strength of 8,500 psi, based on the FDOT SDG Table 1.4.3-1, the FDOT Specs Section 346-3, and the IDS for Index 20010, as shown in Figure 4.6. The reinforcing steel used for the deck and for the Prestressed FIB36 beams' shear reinforcement was ASTM A615, Grade 60 as per the FDOT SDG 1.4.1-B. The shear reinforcement layout at the ends of the FIB36 beams was in accordance with FDOT Index 20036 and the other regions spacing was designed following the beam Elevation details shown in FDOT Index 20036. The prestressing strands used in the FIB36 beams were ASTM A416, Grade 270, low-relaxation in accordance with Section 4.3.1-A of the FDOT SDG. A summary of the materials used in the *SmartBridge* models is shown in Table 4.1 below.

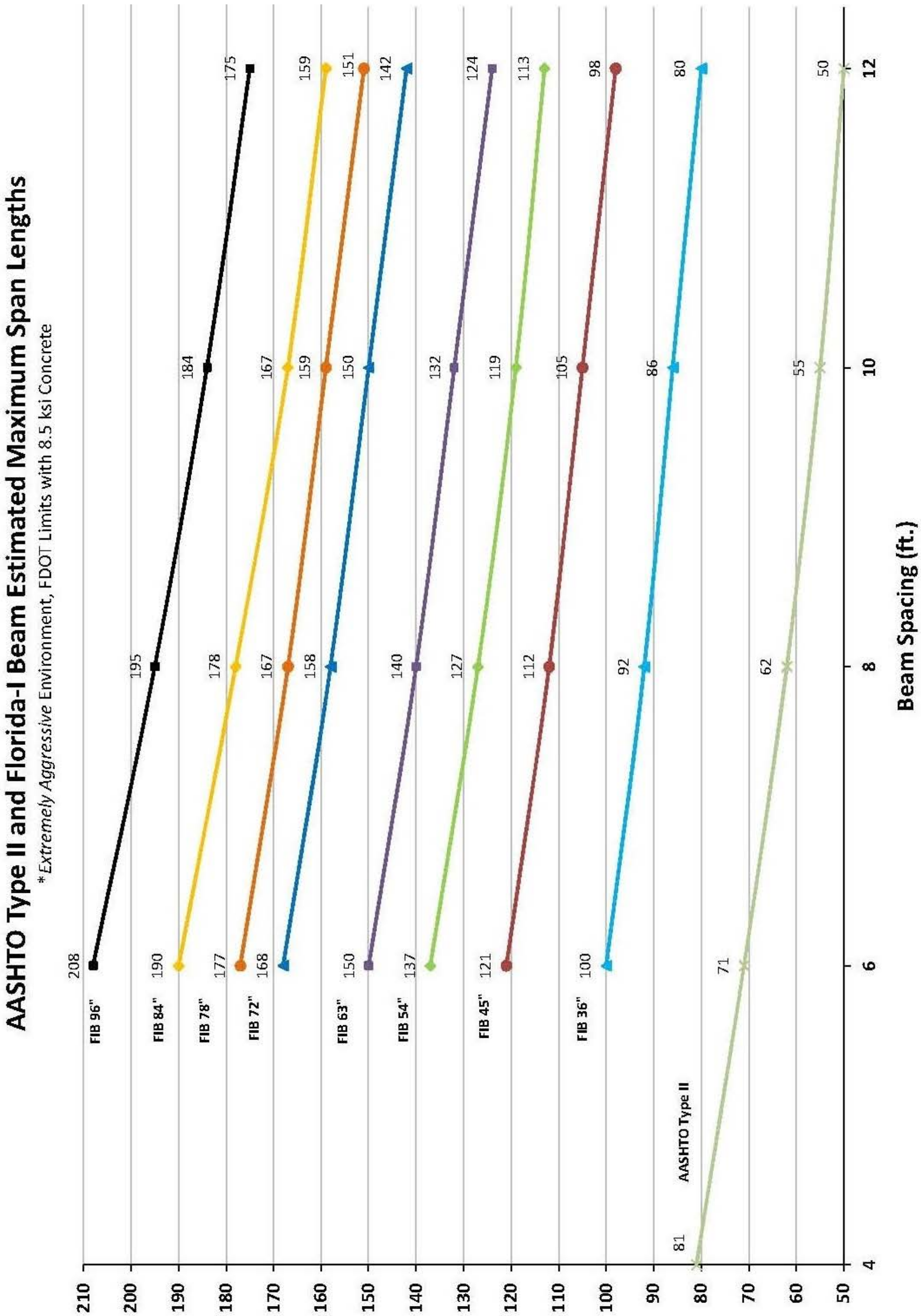


Figure 4.6: FIB36 Beam Maximum span lengths.

Adapted from “IDS 20010” by FDOT, 2014.

Table 4.1: Material properties used in SmartBridge models

Concrete Properties						
ID	Type		Wc (pcf)	fc' (ksi)	Ec (ksi)	Aggr. size (in.)
Concrete 8.5 Beams	Normal		150.00	8.50	5030.40	1.50
Concrete 5.5 Deck	Normal		150.00	5.50	3845.83	1.50
Prestress Steel Properties						
ID	Type		Dia (in.)	Area (in^2)	fpu (ksi)	fy (ksi)	E (ksi)
6/10	Low Relaxation		0.600	0.217	270	243	29000
Reinforcing Steel Properties						
ID	Dia (in.)	Area (in^2)	fy (ksi)	E (ksi)		
#5 Gr60	0.625	0.310	60	29000		
#6 Gr60	0.750	0.440	60	29000		

Each typical section was input as a unit model with 3 different span lengths of 70', 80', and 90'. Then the design was refined to obtain an operating rating factor of 1.4 or greater as required by the FDOT *Bridge Load Rating Manual* (BLRM). Each unit model had a total length of 240' with 3 simply supported spans and Type K typical section based on LRFD Table 4.6.2.2.1-1. The superstructures used in *SmartBridge* are shown in Table 4.2, showing the varying beam spacing, type of section, unit length, number of spans, and continuity.

Table 4.2: Superstructure information used in SmartBridge models

Unit Index	Unit Group	Unit Type	Unit Length (ft)	Number of Spans	Continutity
1	6 foot spacing	PC Girder with Deck (K)	240	3	Simple Supported for DL and LL
2	8 foot spacing	PC Girder with Deck (K)	240	3	Simple Supported for DL and LL
3	10 foot spacing	PC Girder with Deck (K)	240	3	Simple Supported for DL and LL
4	12 foot spacing	PC Girder with Deck (K)	240	3	Simple Supported for DL and LL
5	14 foot spacing	PC Girder with Deck (K)	240	3	Simple Supported for DL and LL

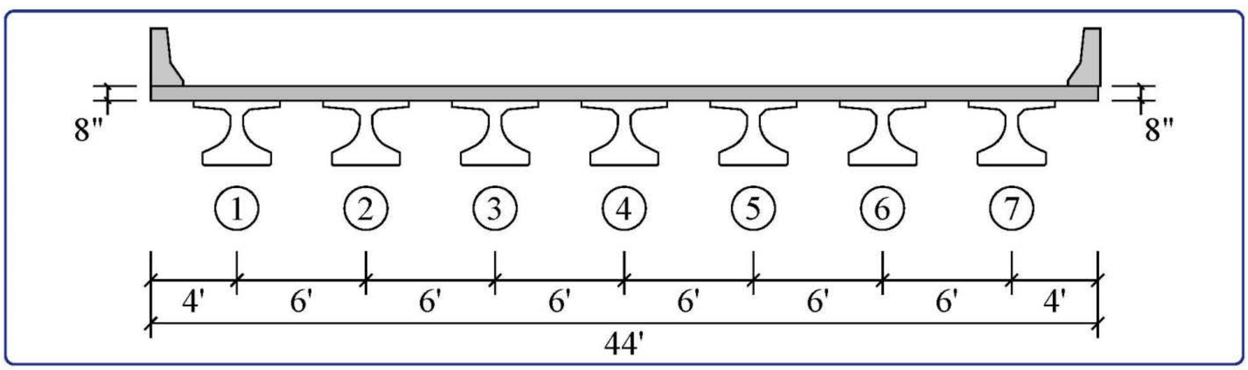
Each superstructure was designed with condition factor of 1.0 and a system factor of 1.0, for new design and section type respectively, based on the FDOT BLRM. Additional loads of 15 psf and

20 psf, for the future wearing surface and the stay-in-place forms respectively, were added in accordance with the FDOT SDG Table 2.2-1, as shown in Table 4.3.

Table 4.3: Unit geometry used in SmartBridge models

Total Length (ft):	240.00	Total Width (ft):	44.00
Condition Factor	1.00	System Factor	1.00
Wearing Surface weight (psf):	15.00	Stay-in-place form Weight (psf):	20.00
Left Overhang (ft):	4.00	Right Overhang (ft):	4.00
Left Curb Width (ft):	1.50	Right Curb Width (ft):	1.50
Lane Width (ft):	12.00	Number of Lanes	3.00
Deck Properties			
Concrete Type	Concrete 5.5 Deck	Structural Thickness (in.)	8.00
Actual Thickness (in.)	8.00		
Spans			
Index	Length (ft)	Left Skew (deg)	Right Skew (deg)
1	70	0	0
2	80	0	0
3	90	0	0

Figures 4.7, 4.8, and 4.9 show the typical section, the prestressing strand layout, de-bonding pattern, and the shear reinforcement layout for the 6-foot beam spacing with a 70-foot span length.



Girder	Girder Type	Spacing (ft)	Bearing2Bearing (ft)	Precast Length (ft)
Girder1	GirderGroup1	N/A	69.00	70.00
Girder2	GirderGroup1	6.00	69.00	70.00
Girder3	GirderGroup1	6.00	69.00	70.00
Girder4	GirderGroup1	6.00	69.00	70.00
Girder5	GirderGroup1	6.00	69.00	70.00
Girder6	GirderGroup1	6.00	69.00	70.00
Girder7	GirderGroup1	6.00	69.00	70.00

Figure 4.7: Typical section used in SmartBridge models

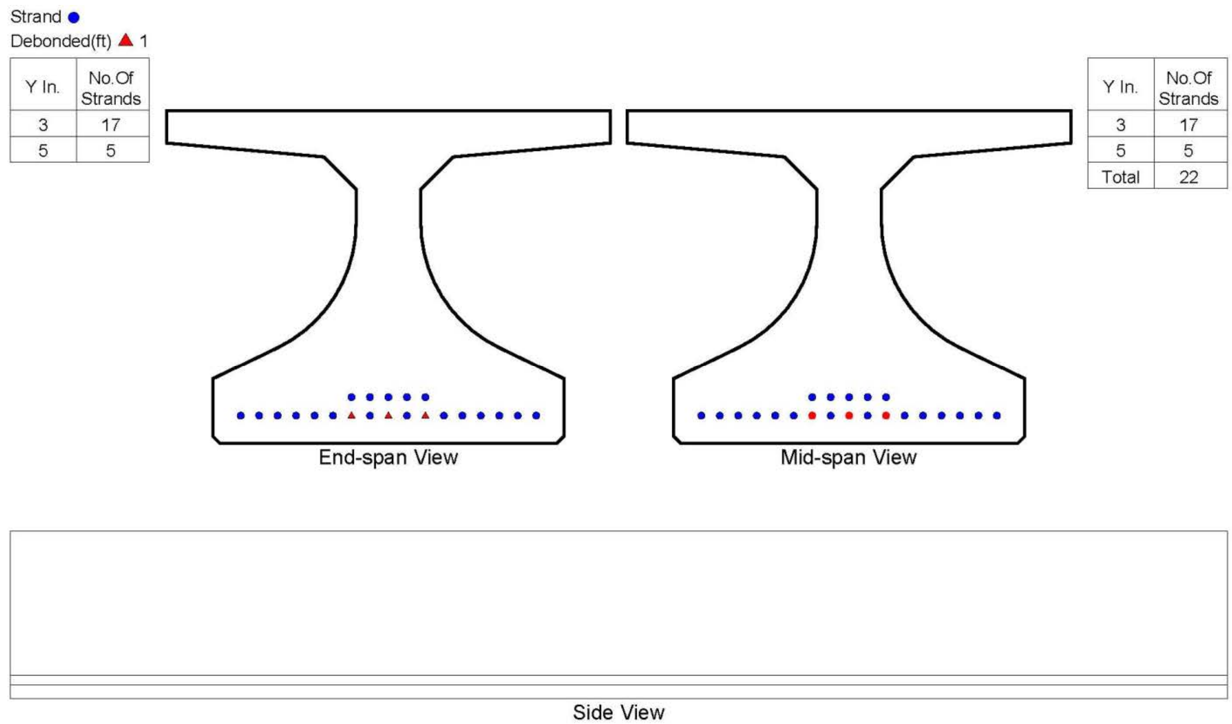
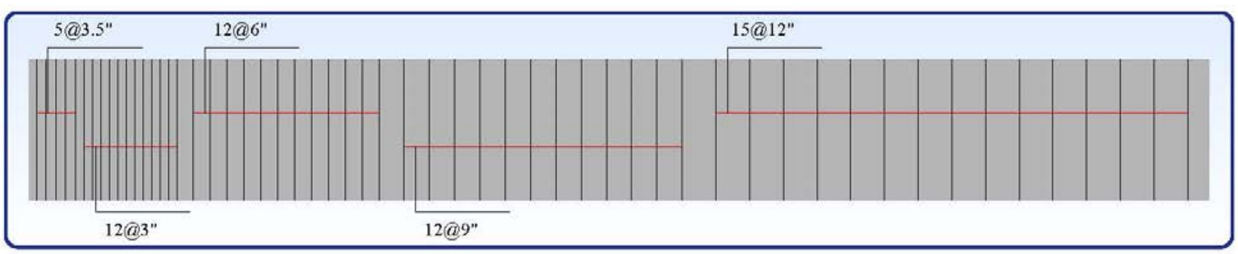


Figure 4.8: Strand layout and debonding pattern used in SmartBridge models



Index	Start X (ft)	End X (ft)	Spacing (in)	No. of Bars	Size	Type
1	0.21	1.38	3.5	5	#6 Gr60	4-leg
2	1.63	4.38	3	12	#5 Gr60	1-leg
3	4.88	10.38	6	12	#5 Gr60	1-leg
4	11.13	19.38	9	12	#5 Gr60	1-leg
5	20.38	34.38	12	15	#5 Gr60	1-leg

Figure 4.9: Shear reinforcement layout used in SmartBridge models

The AASHTO vehicular live load deflection results obtained from the SmartBridge design and the finite element models, showed consistent results as the beam spacing increased by 2 feet. Table 4.4 and Figures 4.10 through 4.14 summarize the live load deflection results.

Table 4.4: LL Deflection

Beam Spacing (FT)	Span Length (FT)	Smart Bridge Design	Finite Element Model
6	70	-0.388	-0.204
6	80	-0.577	-0.280
6	90	-0.814	-0.371
8	70	-0.429	-0.241
8	80	-0.638	-0.325
8	90	-0.899	-0.423
10	70	-0.467	-0.276
10	80	-0.693	-0.372
10	90	-0.977	-0.510
12	70	-0.503	-0.309
12	80	-0.746	-0.417
12	90	-1.051	-0.540
14	70	-0.537	-0.338
14	80	-0.797	-0.457
14	90	-1.122	-0.592

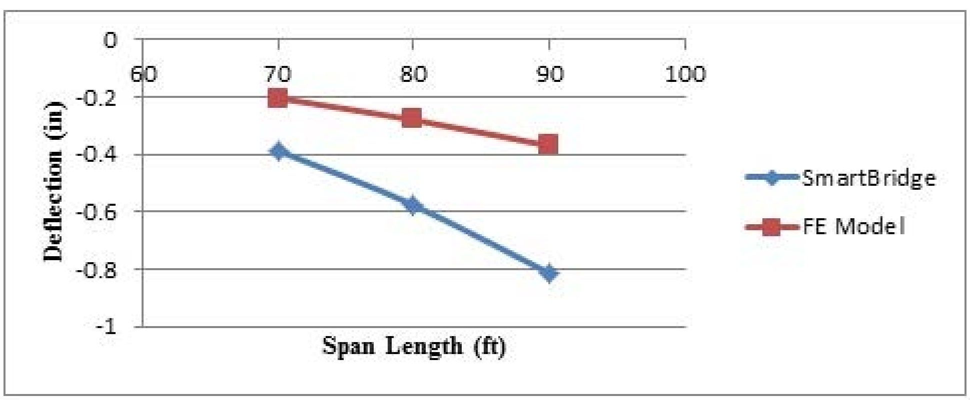


Figure 4.10: LL Deflection for 6-foot beam spacing

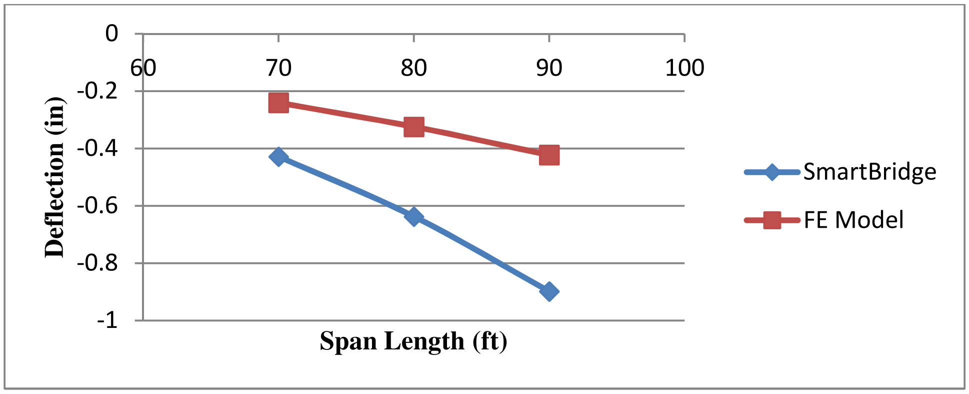


Figure 4.11: LL deflection for 8-foot beam spacing

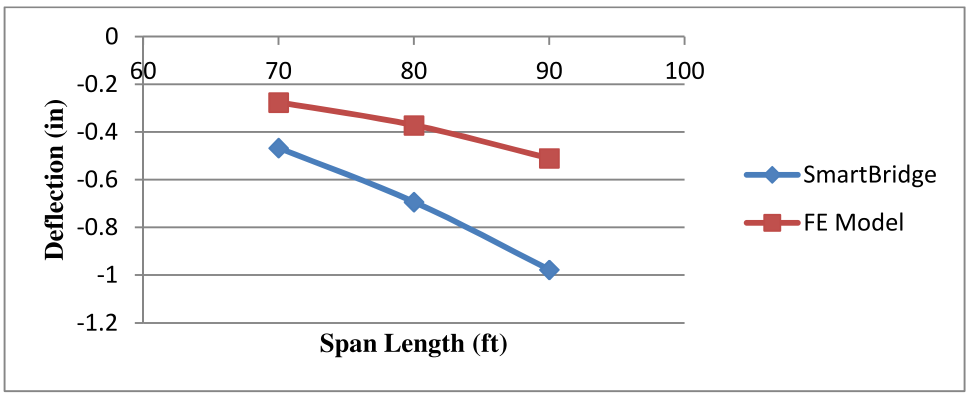


Figure 4.12: LL deflection for 10-foot beam spacing

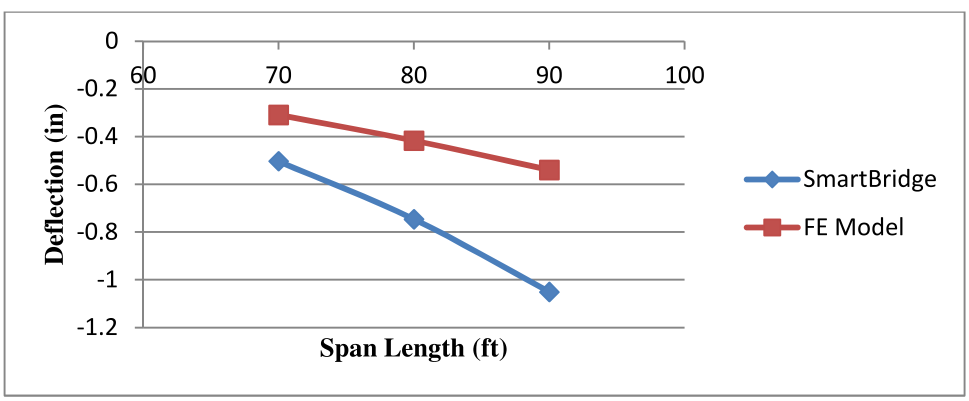


Figure 4.13: LL deflection for 12-foot beam spacing

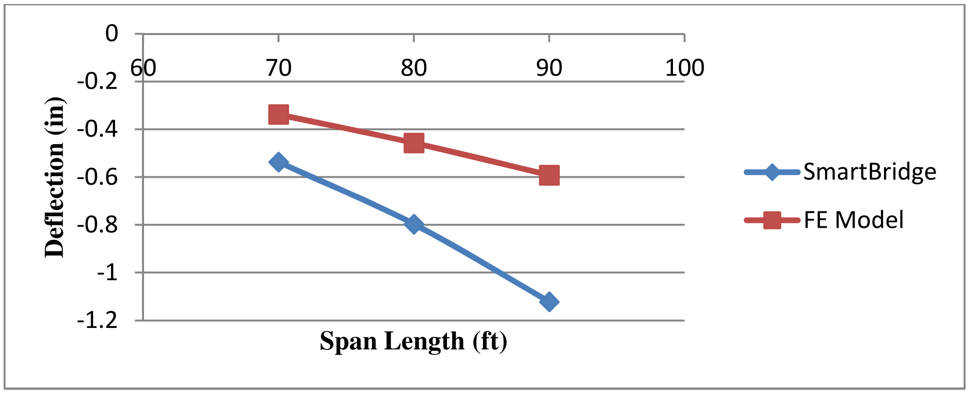


Figure 4.14: LL deflection for 14-foot beam spacing

Empirical Deck Analysis and Design

The 15 bridges were checked to verify whether they meet the empirical method requirements based on the FDOT SDG – January 2009, Section 4.2.4 and the AASHTO LRFD Section 9.7.2.4. The FDOT SDG Section 4.2.4, states that if the structure meets the criteria in LRFD 9.7.2.4, the deck slabs may be designed using the empirical design method with some variation to the reinforcing steel provided in the deck. Instead of a reinforcing area of 0.27 in^2 per foot in the bottom layer and 0.18 in^2 per foot in the top layer, use no. 5 bars at 12 inches in both directions in the top and the bottom layers. This would result in a reinforcing area of 0.31 in^2 per foot in the top and bottom layers.

Since the 8 inch deck thickness did not result in favorable results for the finite element models, thicker decks had to be analyzed based on the beam spacing and span length. The 8-foot, 10-foot, 12-foot, and 14-foot beam spacing models met all the empirical design method conditions with the 8 inch deck thickness and the modified thicker decks. The 6-foot beam spacing bridges did not meet all the conditions of the empirical method. The condition that failed was the ratio of effective length to design depth which required this ratio to be between 6 and 18, as shown in Equation 5. This was due to the larger top flange width of 48 inches of the

FIB36 beams. In order to correct this issue, the deck thickness was reduced to 7" for the 6-foot beam spacing models, as shown in Equation 6.

$$Deck_{DesignDepth} = 8\ in$$

$$Eff\_Len\_to\_Design\_Thick\_Ratio = Deck_{EffLength}/Deck_{DesignDepth} \tag{5}$$

$$Eff\_Len\_to\_Design\_Thick\_Ratio = 5.563$$

$$Deck_{DesignDepth} = 7\ in$$

$$Eff\_Len\_to\_Design\_Thick\_Ratio = Deck_{EffLength}/Deck_{DesignDepth} \tag{6}$$

$$Eff\_Len\_to\_Design\_Thick\_Ratio = 6.357$$

The empirical method conditions and final design are summarized in Appendix A and B.

Traditional Deck Analysis and Design

The deck of the 15 bridge models were designed following the traditional deck design, also known as equivalent strip method, as continuous beams for the negative and positive moments. The exposure condition for crack control was assumed to be Class 1 in accordance with LRFD 5.7.3.4. The negative moments' design section was calculated based on LRFD. The live load design moments were obtained from AASHTO LRFD Appendix A4 shown in Table 3.2. The dead load positive and negative moments due to the deck self-weight and the FDOT Type-F Barrier, and the future wearing surface positive and negative moments were obtained from a *STAAD.Pro V8i* line model, as shown in Figure 4.15.

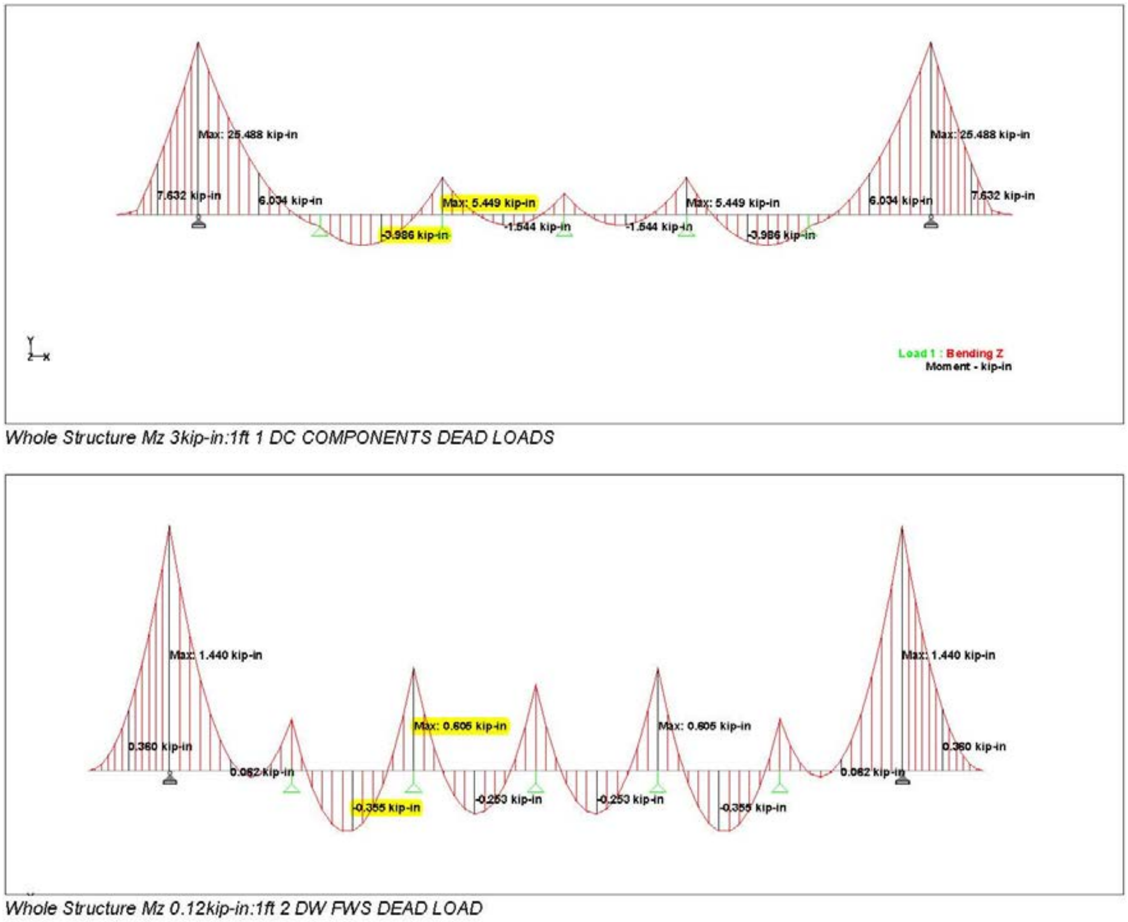


Figure 4.15: *STAAD.Pro V8i* Model showing DC and DW moments for 6-foot beam spacing

The dead load moment and live load moment were combined to calculate the maximum service moment and strength moment, using Equations 7 and 8.

$$M_{service} = \max\left(M_{DC\ Pos} + M_{DW\ Pos} + M_{LL\ Pos}, M_{DC\ Neg} + M_{DW\ Neg} + M_{LL\ Neg}\right)$$

(7)

$$M_{strength} = \max\left(1.25M_{DC\ Pos} + 1.50M_{DW\ Pos} + 1.75M_{LL\ Pos},\right.$$

$$\left.1.25M_{DC\ Neg} + 1.50M_{DW\ Neg} + 1.75M_{LL\ Neg}\right) \quad (8)$$

The flexure reinforcement was designed for the ultimate moment in the deck, which is the minimum of 1.2 x cracking moment and 1.33 x strength moment, as shown in Equation 9.

$$M_u = M_{strength} if\ M_{strength} \geq 1.2\ M_{cr}$$

$$M_u = \min\left(1.33 M_{strength}, 1.2\ M_{cr}\right) otherwise \tag{9}$$

The cracking moment was calculated using Equation 10.

$$M_{cr} = (f_r\ I_g)/y_t \tag{10}$$

Where: M_{cr} = *cracking moment*

f_r = *modulus of rupture* = $0.24\sqrt{f'}_c$

I_g = *moment of inertia* = $b\ h^3/12$

y_t = *dist. from extreme tensile fiber to neutral axis* = $h/2$

Then the section was checked if it was tension or compression controlled to determine the strength reduction factor (□) for the flexural resistance based on LRFD 5.7.2.1 and 5.7.2.2. The moment capacity was checked for the reinforcing steel provided using Equation 11.

$$checkMoment = OK\ if\ \emptyset M_n \geq M_u$$

$$checkMoment = No\ Good\ if\ \emptyset M_n < M_u \tag{11}$$

$$\emptyset M_n = \emptyset_M\ A_{sMain}\ f_y(d_e - a/2)$$

Where: a = *depth of equivalent rectangular stress block* = $(A_s f_y / 0.85 f'_c\ b)$

The stress in the reinforcing steel was also computed at the service limit state using Equation 12.

$$f_{ss} = M_{service}/[A_{sMain}(d_e - x/3) \tag{12}$$

Where: d_e = *dist. from extreme compression fiber to centroid of rebars* =

$$d_e = h - cover - (d_{bar} bar_{main}/2)$$

$$f(x) = (b\,x^2/2A_{sMain}n) + x \ - d_e$$

x = *depth of neutral axis* = $root\ (f(x), x, 0, d_e)$

A sample of the traditional design method is included in Appendix C and D.

Finite Element Modeling

The 15 bridges were analyzed using three-dimensional linear finite element models that include all elements of the structure such as traffic railings, deck, beams, caps, and piles. The decks were modeled as 4-noded (quadrilateral) plates with varying thicknesses based on the beam spacing and span length and also included the thickened slab end based on the FDOT SDG section 4.2.13. The deck plates used were 2'x2' and were generated in *STAAD.Pro V8i* using the mesh generation facility. Figure 4.16 shows the sign convention of the deck plates in *STAAD.Pro V8i* in the local plate coordinate system.

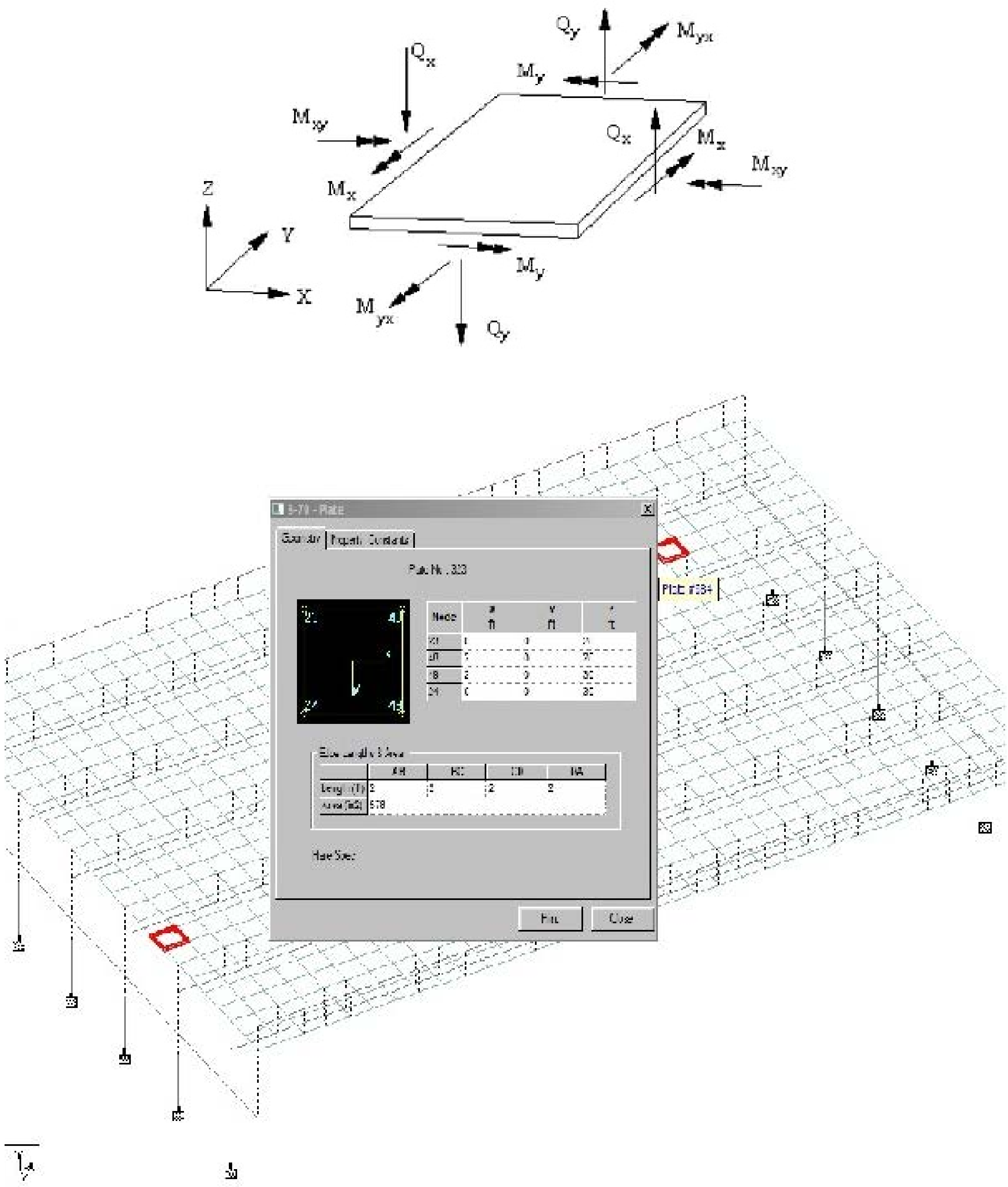


Figure 4.16: Plate sign convention used in *STAAD.Pro V8i*

The FIB36 beams, the FDOT F-Shape barrier, the bent cap, and the piles were modeled as beam/column elements in *STAAD.Pro V8i.* The FIB36 beams and the F-Shape barrier were built as special element using the user defined table with geometry matching the FDOT Design Standards Index 20036 and Index 420 for the FIB36 beams and F-Shape barrier respectively as shown in Figures 4.17 and 4.18.

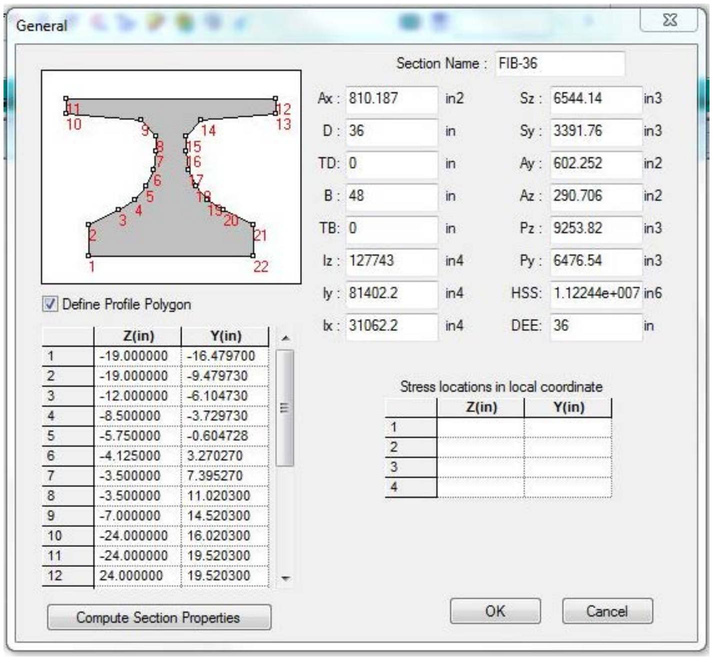


Figure 4.17: FIB36 beam geometry in *STAAD.Pro V8i*

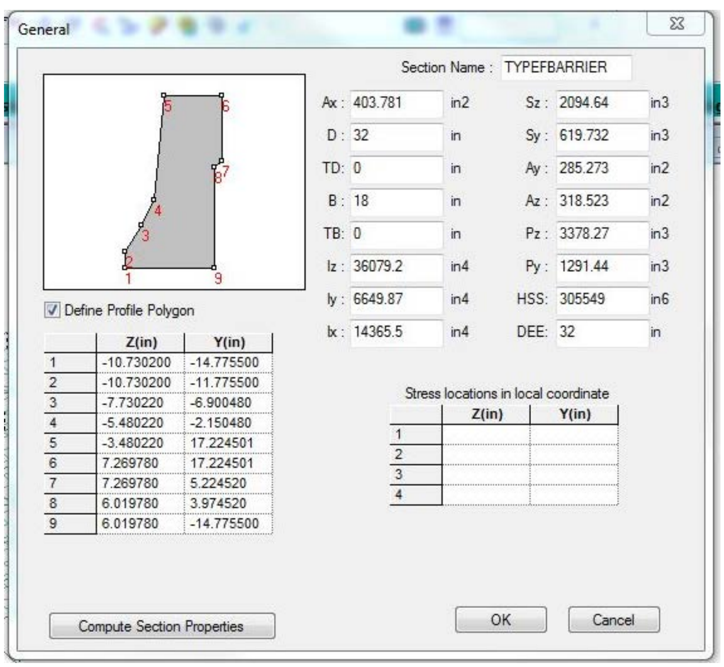


Figure 4.18: F-Shape barrier geometry in *STAAD.Pro V8i*

The dead loads applied in the finite element models included the selfweight of all the element of the structure. The stay-in-place forms load was applied as a uniform pressure between the beams' top flanges while the future wearing surface load was applied as a uniform pressure between the curb lines. In order to obtain the maximum live load effect, the HL-93 design truck was modeled as a moving load on the deck surface in the longitudinal and transverse directions at one foot increments.

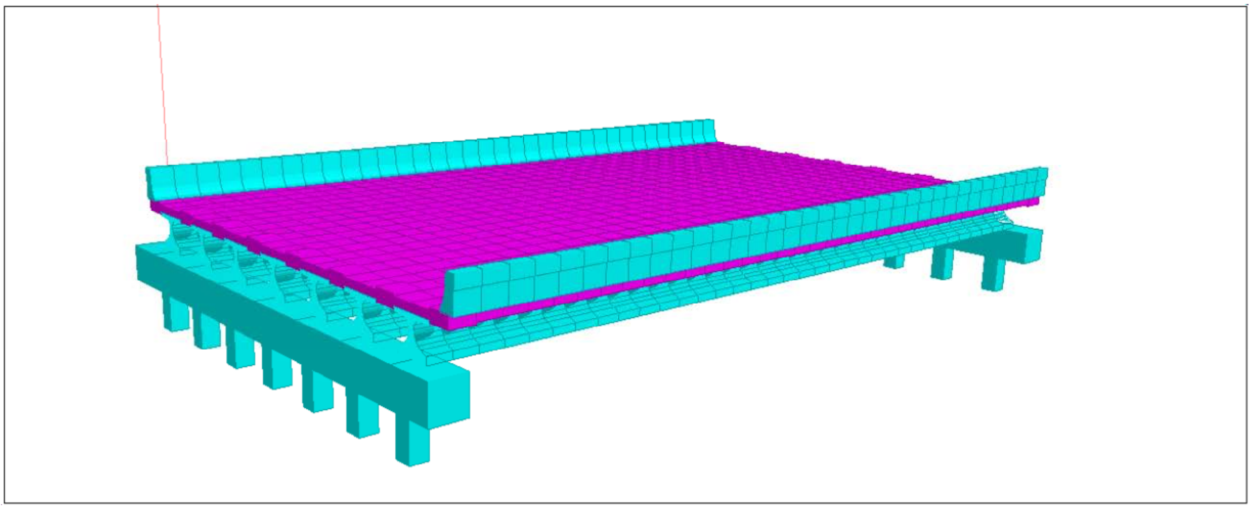


Figure 4.19: Three-dimensional finite element model

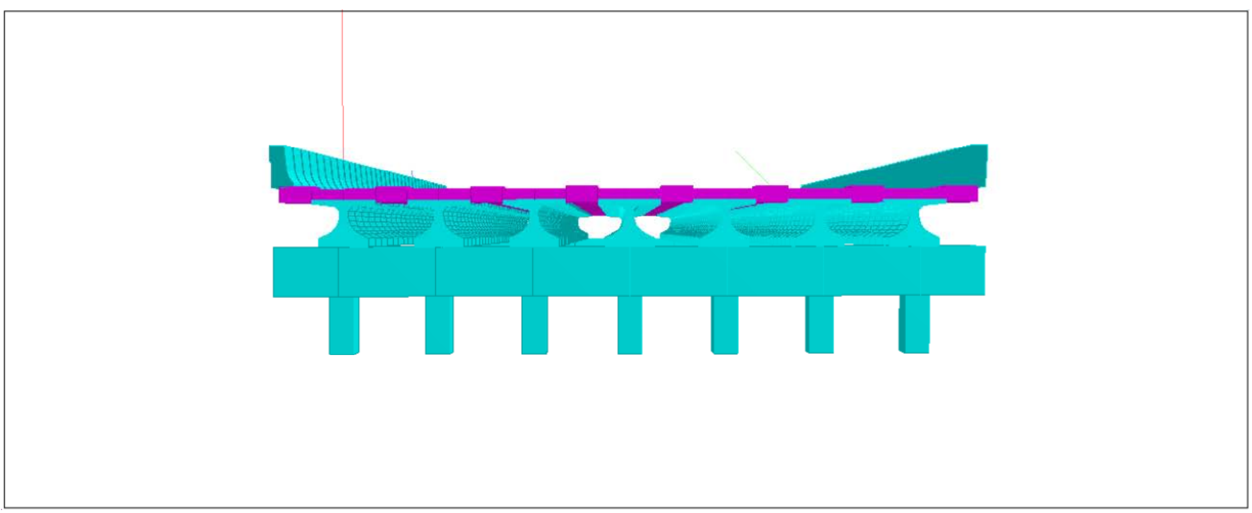


Figure 4.20: Typical section of three-dimensional finite element model

The service moments due to live load, dead loads (including stay-in-place forms) and future wearing surface were output in *STAAD.Pro V8i* separately and then imported to the Mathcad sheet to design the flexure reinforcement in the deck, check the deck for cracking and check the tensile stresses in the reinforcing steel.

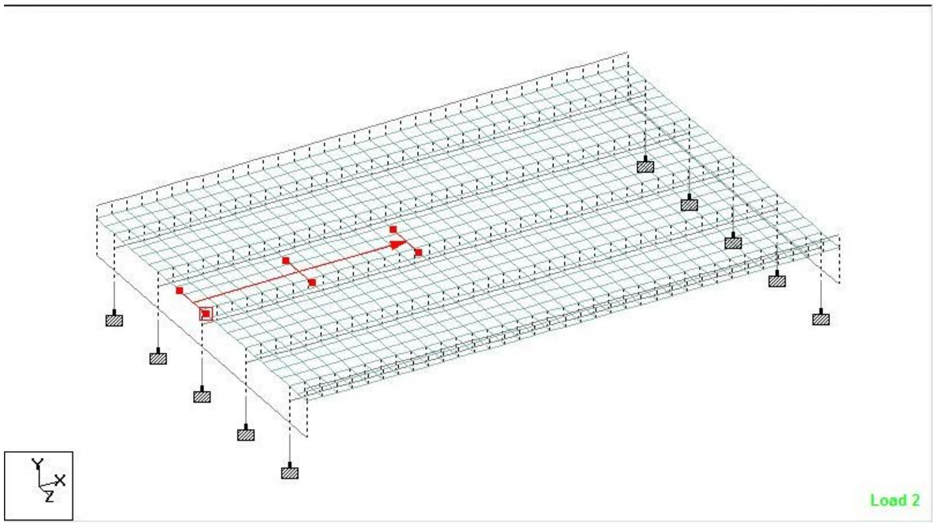


Figure 4.21: Finite element model showing HL-93 moving load on deck

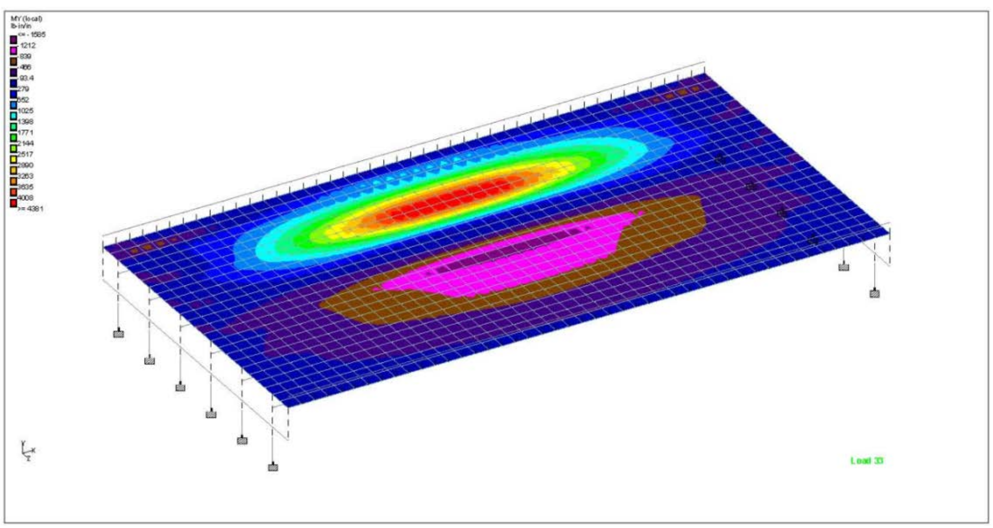


Figure 4.22: Finite element model showing maximum positive moment in the deck

Interpretation of Results

Initially all 15 models were analyzed with 8” thick deck based on the FDOT SDG Section 4.2 requirements which states that for new bridge construction of short bridges, the deck thickness shall be a minimum of 8 inches for cast-in-place concrete with no sacrificial surface deduction. Short bridges are defined as having a Profile Grade Length, between front faces of backwalls, of less than or equal to 100 feet. The main reinforcing steel ratio (ρ) required was summarized in Table 4.5, and showed that the reinforcing steel ratio obtained from the finite element models was well above the 0.454% (for No. 5 at 12”) set by the FDOT SDG for the empirical design with deck thickness of 8 inches, except for the case of 8-foot beam spacing with 70-foot span length.

Table 4.5: Required main reinforcing steel ratio for 8” thick decks

Required Main Reinforcing Steel Ratio / Layer (ρ)							
Beam Spacing (FT)	Span Length (FT)	Deck Thickness (in)	Overhang (FT)	Bar Size	$\rho_{required}$: Traditional Design	$\rho_{required}$: Empirical Design	$\rho_{required}$: Finite Element Design
6	70	8.00	4	No. 5	0.530%	No Good*	0.424%
	80	8.00	4	No. 5	0.530%	No Good*	0.424%
	90	8.00	4	No. 5	0.530%	No Good*	0.488%
8	70	8.00	4	No. 5	0.634%	0.454%	0.439%
	80	8.00	4	No. 5	0.634%	0.454%	0.513%
	90	8.00	4	No. 5	0.634%	0.454%	0.584%
10	70	8.00	4	No. 5	0.777%	0.454%	0.498%
	80	8.00	4	No. 5	0.777%	0.454%	0.583%
	90**	8.00	4	No. 5	0.777%	0.454%	0.665%
12	70	8.00	4	No. 5	0.931%	0.454%	0.529%
	80	8.00	4	No. 5	0.931%	0.454%	0.628%
	90**	8.00	4	No. 5	0.931%	0.454%	0.727%
14	70**	8.00	4	No. 5	1.065%	0.454%	0.528%
	80**	8.00	4	No. 5	1.065%	0.454%	0.655%
	90**	8.00	4	No. 5	1.065%	0.454%	0.760%

* Does not meet all empirical design conditions
** Falls outside the FDOT IDS 20010 limitations

The average reinforcing steel ratio (ρ), for the 8-inch thick decks, was plotted versus the beam spacing and showed that the required reinforcing steel ratio obtained from the finite element models is between the required reinforcing steel ratios obtained from the traditional method and the empirical method, as shown in Figure 4.23.

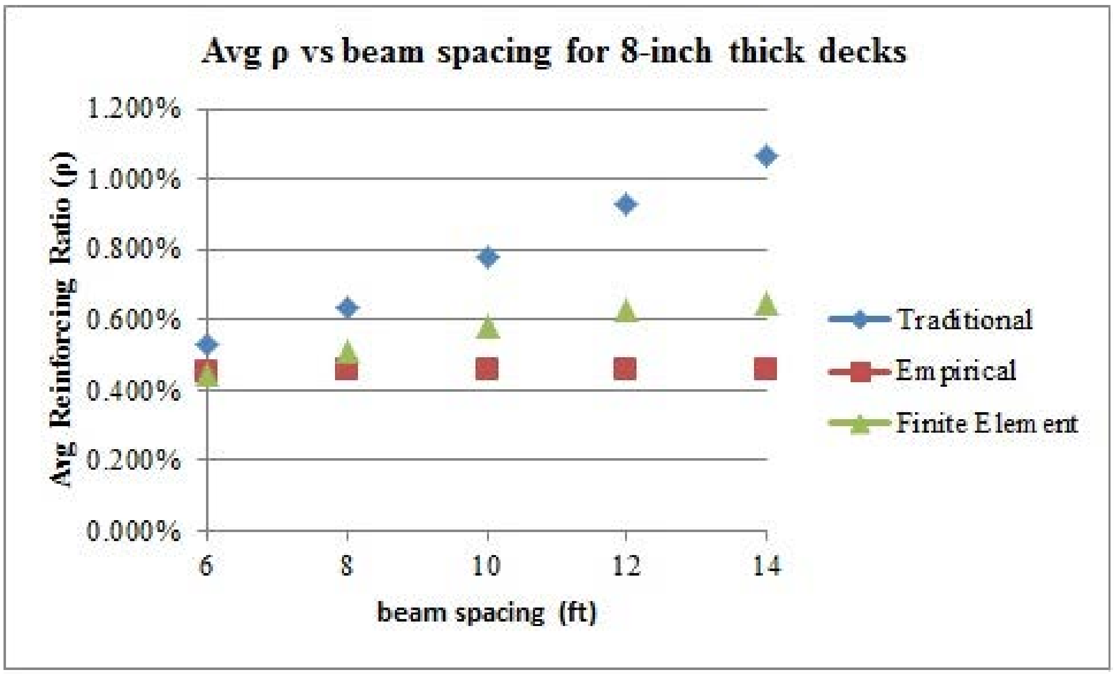


Figure 4.23: Average (ρ) vs beam spacing for 8-inch thick decks

In order to improve the results and obtain lower reinforcing steel ratio in each layer, the deck thicknesses had to be modified. The deck thickness was changed based on two variables; the beam spacing and the span length. Starting with the 6-foot beam spacing, the deck had to be reduced to 7.0” to meet the AASHTO empirical design conditions. For the 8-foot beam spacing, with span lengths of 70’, 80’ and 90’, the deck thickness was 9.0”, 9.0”, and 9.5” respectively with a reinforcing ratio of 0.334%, 0.334%, and 0.330% per layer. For the 10-foot beam spacing, with span lengths of 70’, 80’, and 90’, the deck thickness was 9.0”, 9.5”, and 10.0” respectively with a reinforcing ratio of 0.334%, 0.331%, and 0.342% per layer. For the 12-foot beam spacing, with span lengths of 70', 80', and 90', the deck thickness was 9.0", 9.5", and 10.0" respectively with a reinforcing ratio of 0.339%, 0.360%, and 0.381% per layer. For the 14-foot beam spacing, with span lengths of 70', 80', and 90', the deck thickness was 9.5", 10.0", and 10.0" respectively with a reinforcing ratio of 0.325%, 0.340%, and 0.400% per layer. The main reinforcing steel spacing provided is summarized in Table 4.6.

Table 4.6: Main reinforcing steel spacing for varying deck thickness

Main Reinforcing Steel Spacing Provided (inches) / Layer (Spa_{main})							
Beam Spacing (FT)	Span Length (FT)	Deck Thickness (in)	Overhang (FT)	Bar Size	Spa_{main}: Traditional Design	Spa_{main}: Empirical Design	Spa_{main}: Finite Element Design
6	70	7.00	4	No. 5	9.0	12.0	12.0
	80	7.00	4	No. 5	9.0	12.0	12.0
	90	7.00	4	No. 5	9.0	12.0	12.0
8	70	9.00	4	No. 5	8.5	12.0	12.0
	80	9.00	4	No. 5	8.5	12.0	12.0
	90	9.50	4	No. 5	8.5	12.0	12.0
10	70	9.00	4	No. 5	7.0	12.0	12.0
	80	9.50	4	No. 5	7.0	12.0	12.0
	90**	10.00	4	No. 5	7.0	12.0	11.0
12	70	9.00	4	No. 5	5.5	12.0	12.0
	80	9.50	4	No. 5	5.5	12.0	11.0
	90**	10.00	4	No. 5	5.5	12.0	10.0
14	70**	9.50	4	No. 5	5.0	12.0	12.0
	80**	10.00	4	No. 5	5.0	12.0	11.0
	90**	10.00	4	No. 5	5.0	12.0	9.0

** Falls outside the FDOT IDS 20010 limitations

The main reinforcing steel ratio was calculated for the three design methods and summarized in Table 4.7 and showed that the reinforcing steel ratios obtained from the finite element model agree for the most part with the reinforcing steel ratio obtained from the empirical method.

Table 4.7: Required main reinforcing steel ratio for varying deck thickness

Required Main Reinforcing Steel Ratio / Layer (ρ)							
Beam Spacing (FT)	Span Length (FT)	Deck Thickness (in)	Overhang (FT)	Bar Size	$\rho_{required}$: Traditional Design	$\rho_{required}$: Empirical Design	$\rho_{required}$: Finite Element Design
6	70	7.00	4	No. 5	0.530%	0.498%	0.389%
	80	7.00	4	No. 5	0.530%	0.498%	0.415%
	90	7.00	4	No. 5	0.530%	0.498%	0.475%
8	70	9.00	4	No. 5	0.634%	0.359%	0.334%
	80	9.00	4	No. 5	0.634%	0.359%	0.334%
	90	9.50	4	No. 5	0.634%	0.336%	0.330%
10	70	9.00	4	No. 5	0.777%	0.359%	0.334%
	80	9.50	4	No. 5	0.777%	0.336%	0.331%
	90**	10.00	4	No. 5	0.777%	0.316%	0.342%
12	70	9.00	4	No. 5	0.931%	0.359%	0.339%
	80	9.50	4	No. 5	0.931%	0.336%	0.360%
	90**	10.00	4	No. 5	0.931%	0.316%	0.381%
14	70**	9.50	4	No. 5	1.065%	0.336%	0.325%
	80**	10.00	4	No. 5	1.065%	0.316%	0.340%
	90**	10.00	4	No. 5	1.065%	0.316%	0.400%

** Falls outside the FDOT IDS 20010 limitations

The average reinforcing steel ratio (ρ), for the decks with other thicknesses, was plotted versus the beam spacing and showed that the required reinforcing steel ratio obtained from the finite element models converges with the required reinforcing steel ratios obtained from the empirical method, as shown in Figure 4.24.

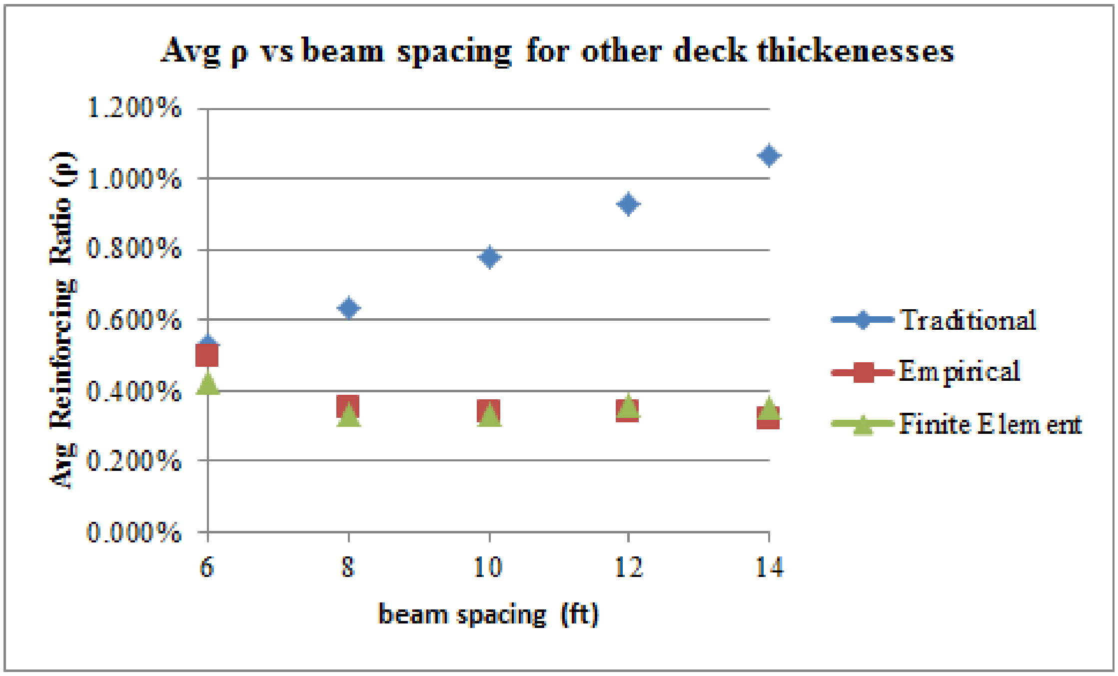


Figure 4.24: Average (ρ) vs beam spacing for other deck thicknesses

The cracking moment in the decks were calculated at service load and compared to the allowable cracking moment calculated in Equation 10. Based on the reinforcing steel provided in the decks, none of the deck showed cracking at service loads. The tensile stress in the concrete deck at service was obtained for the *STAAD.Pro V8i* model plate element stresses. Then it was compared to the allowable concrete modulus of rupture. The tensile stress in the reinforcing steel at service limit state was also calculated in the Mathcad sheet and then compared to the allowable stress in the rebars at operating level which is 36.0 ksi. The results showed that none of the decks analyzed in the finite element models showed any overstressing of the concrete in tension or overstressing of reinforcing steel in tension.

Table 4.8: Deck cracking check at service

Check Cracking in Concrete Deck at Service Limit State for Finite Element Models ($M_{service}$ < M_{cr} => OK)						
Beam Spacing (FT)	Span Length (FT)	Deck Thickness (in)	$M_{service}$: Service Moment (kip. ft/ft)	$M_{cracking}$: Cracking Moment (kip. ft/ft)	$M_{service}$/ $M_{cracking}$	Check Moment
6	70	7.00	2.939	4.597	0.64	OK
	80	7.00	3.476	4.597	0.76	OK
	90	7.00	3.975	4.597	0.86	OK
8	70	9.00	4.564	7.598	0.60	OK
	80	9.00	5.356	7.598	0.70	OK
	90	9.50	6.266	8.466	0.74	OK
10	70	9.00	5.282	7.598	0.70	OK
	80	9.50	6.333	8.466	0.75	OK
	90**	10.00	7.527	9.381	0.80	OK
12	70	9.00	5.739	7.598	0.76	OK
	80	9.50	7.010	8.466	0.83	OK
	90**	10.00	8.507	9.381	0.91	OK
14	70**	9.50	6.347	8.466	0.75	OK
	80**	10.00	7.659	9.381	0.82	OK
	90**	10.00	9.023	9.381	0.96	OK

** Falls outside the FDOT IDS 20010 limitations

Table 4.9: Concrete tensile stress check at service

Check Tensile Stress (f_t) in Concrete Deck at Service Limit State for Finite Element Models ($f_t < f_r$) (AASHTO LRFD)						
Beam Spacing (FT)	Span Length (FT)	Deck Thickness (in)	f_t: Tensile Stress (psi)	f_r: 0.24 √f'c Modulus of Rupture (psi)	f_t / f_r	Check stress
6	70	7.00	157.934	562.85	0.28	OK
	80	7.00	172.305	562.85	0.31	OK
	90	7.00	187.882	562.85	0.33	OK
8	70	9.00	170.760	562.85	0.30	OK
	80	9.00	185.456	562.85	0.33	OK
	90	9.50	197.189	562.85	0.35	OK
10	70	9.00	198.263	562.85	0.35	OK
	80	9.50	216.109	562.85	0.38	OK
	90**	10.00	247.087	562.85	0.44	OK
12	70	9.00	223.140	562.85	0.40	OK
	80	9.50	245.079	562.85	0.44	OK
	90**	10.00	262.523	562.85	0.47	OK
14	70**	9.50	245.008	562.85	0.44	OK
	80**	10.00	271.019	562.85	0.48	OK
	90**	10.00	291.832	562.85	0.52	OK

** Falls outside the FDOT IDS 20010 limitations

Table 4.10: Reinforcing steel tensile stress check at service

Check Tensile Stress in Reinforcing Steel at Service Limit State (f_{ss}) for Finite Element Models (f_{ss} < 36 ksi for Grade 60 Rebars) (AASHTO MBE)						
Beam Spacing (FT)	Span Length (FT)	Deck Thickness (in)	f_{ss}: Rebars Tensile Stress (ksi)	f_{all}: Rebars Allowable Stress (ksi)	f_{ss} / f_{all}	Check stress
6	70	7.00	23.883	36.00	0.66	OK
	80	7.00	28.247	36.00	0.78	OK
	90	7.00	32.302	36.00	0.90	OK
8	70	9.00	26.457	36.00	0.73	OK
	80	9.00	31.048	36.00	0.86	OK
	90	9.50	33.885	36.00	0.94	OK
10	70	9.00	30.619	36.00	0.85	OK
	80	9.50	34.247	36.00	0.95	OK
	90**	10.00	35.061	36.00	0.97	OK
12	70	9.00	33.268	36.00	0.92	OK
	80	9.50	34.850	36.00	0.97	OK
	90**	10.00	35.790	36.00	0.99	OK
14	70**	9.50	34.323	36.00	0.95	OK
	80**	10.00	35.676	36.00	0.99	OK
	90**	10.00	34.626	36.00	0.96	OK

** Falls outside the FDOT IDS 20010 limitations

The results obtained from the developed FE models indicated that the reinforcing steel ratio is closer to that of the empirical method and much lower than that of the traditional method. Similar results were obtained from previous studies and tests on deck reinforcement. These studies also indicated that the decks are over designed with high steel reinforcement ratios with safety factor of 10.0 when using the traditional method and a safety factor of 8.0 when using the empirical method (AASHTO LRFD). Tests conducted for the Michigan DOT showed that the stress levels in the reinforcing steel were very low for decks designed using the empirical method (Nowak et al., 2003) which also confirms the feasibility of the finite element models analyzed in this study.

Conclusions and Recommendations

Based on the findings of this report, the 15 bridge models were analyzed successfully with the three-dimensional linear finite element software with moving live loads. The initial results, when using the 8-inch thick concrete plate elements, showed that the reinforcing steel ratio obtained from the finite element models was well above the 0.454% (for No. 5 at 12") set by the FDOT SDG for the empirical design except for the case of 8-foot beam spacing with 70-foot span length.

The deck thicknesses for the finite element models were then modified to reduce the reinforcing steel ratio. That resulted in the reinforcing steel ratios obtained from the finite element model agreeing for the most part with the reinforcing steel ratio obtained from the empirical method. The models that had slightly higher reinforcing steel ratio from the finite element method than the empirical method were the ones that did not meet the FDOT IDS 20010 limitations for beam spacing and span length when using the FIB36 beams.

Conclusions

The following are the conclusions based on this study:

- The finite element models showed the feasibility of using empirical design method, when using thicker decks (>8").
- The reinforcing steel ratio obtained from the finite element method matched the reinforcing steel ratio obtained from the empirical method for the most part.
- The results obtained from the finite element method showed that the empirical design is an acceptable and solid method that should provide sufficient reinforcing steel to control cracking in the reinforced concrete decks.

- The empirical deck design method could be considered for future use on FDOT bridge decks since it would result in a more economical design than the traditional method.
- FDOT should consider using thicker decks for beam spacing between 8 and 14 feet for the empirical method. Other States have used the empirical design method successfully with thicker decks: Idaho: 10” with beam spacing of 13.5’, Louisiana: 9.5” with beam spacing of 12.0’, New Jersey: 10.75” with beam spacing of 13.0’, and New York: 9.5” for any beam spacing.
- The success of using the empirical design method could result in material saving by using less amount of reinforcing steel in the reinforced concrete decks as shown in the reinforcing steel ratios results.

Recommendations

The following are recommendations presented to FDOT for future work:

- FDOT should consider testing full scale bridges to verify the reinforcing steel ratio results obtained from this study. The full scale bridges could consist of four options, as follows:
    - Span length of 45.0’ and beam spacing of 10.0’
    - Span length of 45.0’ and beam spacing of 12.0’
    - Span length of 60.0’ and beam spacing of 10.0’
    - Span length of 60.0’ and beam spacing of 12.0’

- FDOT should investigate creating a deck design standard for decks that meet the empirical design conditions set by the AASHTO LRFD.

- Since this study was limited to the FIB36 beams, FDOT should consider conducting additional research/analysis using other FIB beams with longer spans to verify the reinforcing steel ratios in the concrete decks. The span length and beam spacing should be based on the FDOT IDS 20010.

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Appendix A: Empirical design method sample, 6-foot beam spacing

EMPIRICAL DECK DESIGN

Codes and Specification

- "AASHTO LRFD Bridge Design Specifications "American Association of State Highway and Transportation Officials (AASHTO), 6th Edition, 2012 with Interims through 2013.
- "Florida Department of Transportation Structures Design Guidelines for Load and Resistance Factor Design," January 2013 Edition.
- "Florida Department of Transportation Design Standards", 2014.

Unit Definitions:

$pcf := \frac{lbf}{ft^3}$ $\quad ksi := 1000psi$ $\quad kip := 1000 \cdot lbf$

Reinforcing Dimensions

Deck Design:

Bridge Length Definitions **(SDG 4.2.1)**

For establishing profilograph and deck thickness requirements, bridge structures are defined as Short Bridges or Long Bridges. The determining length is the length of the bridge structure measured along the Profile Grade Line (PGL) from front face of backwall at Begin Bridge to front face of backwall at End Bridge of the structure. Based upon this established length, the following definitions apply:

A. Short Bridges: Bridge structures less than or equal to 100 feet in PGL length.

B. Long Bridges: Bridge structures more than 100 feet in PGL length.

Input

(NOTE: For deck slabs, use same reinforcement Top & Bot. Do not include Integral Wearing Surface in "h" OR "cover")

Variables:

Beam Type:	Beam := "FIB36"	(Use 2, 3, 4, 5, 6, FBT72, FBT78, FIB36, FIB45, FIB54, FIB63, FIB72, FIB78)
Beam Spacing:	$S_{beam} := 6ft + 0.0in$	
Overhang:	$Overhang := 4ft + 0.0in$	
Begin Bridge Station:	$Begin_{sta} := 0ft$	
End Bridge Station:	$End_{sta} := 70ft$	
Skew Angle:	$Skew_{angle} := 0 \cdot deg$	

$PGL\_Length := (End_{sta} - Begin_{sta}) = 70 ft$

Bridge_Def := if(PGL_Length ≤ 100ft, "Short Bridge", "Long Bridge") Bridge_Def = "Short Bridge"

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EMPIRICAL DECK DESIGN

Deck Thickness Determination:

T_Beams := "NO"   New_Const := "YES"

Deck_Thickness := if(T_Beams = "YES" ,6.5in,if(New_Const = "YES" ,if(Bridge_Def = "Short Bridge" ,7in,8.5in),"Check widening"))

Deck_Thickness = 7·in

$\text{Deck}_{\text{DesignDepth}}$:= Deck_Thickness

Beam Properties

Constants:

Top Flange Width: $b_{tf} = 48 \cdot in$

Web Width: $t_w = 7 \cdot in$

$\text{Deck}_{\text{EffectiveLength}} := (S_{beam} - t_w) - (b_{tf} - t_w) \cdot 0.5$ LRFD 9.7.2.3 for definition of effective length

$\text{Deck}_{\text{EffectiveLength}} = 3.708\ ft$

Deck Design (SDG 4.2.4)

Empirical Design Method: For Category 1 structures meeting the criteria in LRFD [9.7.2.4] and are not subject to either staged construction or future widening, design deck slabs by the Empirical Design method of LRFD [9.7.2]. In lieu of the reinforcing requirements of LRFD [9.7.2.5], use no. 5 bars at 12-inch centers in both directions in both the top and bottom layers. Place two additional No. 5 bars between the primary transverse top slab bars (4-inch nominal spacing) in the slab overhangs to meet the TL-4 loading requirements for the FDOT standard traffic railings. Extend one of the additional bars to the midpoint between the exterior beam and the first interior beam; extend the second additional bar 36-inches beyond this midpoint. The maximum deck overhang is 6 feet, measured from the centerline of the exterior beam.

Skew Reinforcement: (SDG 4.2.11) (LRFD 9.7.1.3)

Transverse_Skew_Reinf_Placement := if($\text{Skew}_{\text{angle}}$ < 15deg,"Parallel to skew entire slab length" ,"Prependicular_to_CL_span")

Transverse_Skew_Reinf_Placement = "Parallel to skew entire slab length"

Additional_Longitudinal_Reinf := if($\text{Skew}_{\text{angle}}$ < 15deg,"Not Needed" ," #5 @6" along span -both ends")

Additional_Longitudinal_Reinf = "Not Needed"

Additional_Trans_Reinf := if($\text{Skew}_{\text{angle}}$ < 15deg,"Not needed" ," Three #5 bars @6", full width,parallel to skew, top mat,each end ")

Additional_Trans_Reinf = "Not needed"

EMPIRICAL DECK DESIGN

Conditions to be satisfied to use the Empirical Design Method; **LRFD [9.7.2]**

0- Structure is category 1 as defined in PPM 26.3.1 $Condition_0 := "YES"$

1- Diaphragms used throughout the cross-section at line of support: $Condition_1 := "YES"$

2- Supporting Components are made of steel and/or concrete $Condition_2 := "YES"$

3- Deck is fully cast in place and water cured $Condition_3 := "YES"$

4- Deck is of uniform depth except for haunches at girder flanges and other local thickening: $Condition_4 := "YES"$

5- Ratio of effective length to design depth:

$Deck_{DesignDepth} = 7 \cdot in$

$$Eff\_Len\_to\_Design\_Thick\_Ratio := \frac{Deck_{EffectiveLength}}{Deck_{DesignDepth}}$$

$Eff\_Len\_to\_Design\_Thick\_Ratio = 6.357$

$Condition_5 := if(18 \geq Eff\_Len\_to\_Design\_Thick\_Ratio \geq 6, "YES", "NO")$ $Condition_5 = "YES"$

6- Slab core depth:

$Deck\_Thickness = 7 \cdot in$

$cover := 1.5in$

$slab\_core\_depth := Deck\_Thickness - 2 \cdot cover$ $slab\_core\_depth = 4 \cdot in$

$Condition_6 := if(slab\_core\_depth \geq 4in, "YES", "NO")$ $Condition_6 = "YES"$

7- Effective Length:

$Condition_7 := if\left(Deck_{EffectiveLength} \leq 13.5ft, "YES", "NO"\right)$ $Condition_7 = "YES"$

8- Minimum slab depth:

$Deck\_Thickness = 7 \cdot in$

$Condition_8 := if(Deck\_Thickness \geq 7in, "YES", "NO")$ $Condition_8 = "YES"$

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EMPIRICAL DECK DESIGN

9- Overhang beyond the CL of the outer girder:

$\text{Deck\_Thickness} = 7 \cdot \text{in}$

$\text{Condition}_9 := \text{if}(\text{Overhang} \geq 3 \cdot \text{Deck\_Thickness}, \text{"YES"}, \text{"NO"})$

(Structurally continuous concrete barrier is made composite with the overhang)

$\text{Condition}_9 = \text{"YES"}$

10- The specified 28-day strength of the concrete:

$f_c := 4.5\text{ksi}$

$\text{Condition}_{10} := \text{if}(f_c \geq 4\text{ksi}, \text{"YES"}, \text{"NO"})$

$\text{Condition}_{10} = \text{"YES"}$

11 - The deck is made composite with the supporting structural components:

$\text{Condition}_{11} := \text{"YES"}$

Final Condition Check:

$j := 0..11$

$\text{TotCheck}_j := \text{if}(\text{Condition}_j = \text{"YES"}, 0, 1)$

$\text{TotalCheck} := \text{if}(\max(\text{TotCheck}) = 0, \text{"OK"}, \text{"No Good, one or more checks failed."})$

$\text{TotalCheck} = \text{"OK"}$

These are the failing sections if Total Check is not OK.

$\text{if}(\text{TotalCheck} = \text{"OK"}, \text{"All Good"}, \text{match}(1, \text{TotCheck})) = \text{"All Good"}$

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EMPIRICAL DECK DESIGN

Temperature and Shrinkage Reinforcement (SDG 4.2.12)

For all cast in place decks, design temperature and shrinkage reinforcement per ***LRFD*** **[5.10.8]** except do not exceed 12 inch spacing and the minimum bar size is No. 4.

From LRFD 5.10.8:

For bars or welded wire fabric, the area of reinforcement per foot, on each face and in each direction, shall satisfy:

$$\text{Area\_of\_Steel} \ge \frac{1.30bh}{2(b+h)fy}$$ LRFD Eq 5.10.8-1

$$0.11in^2 \le \text{Area\_of\_Steel} \le 0.60in^2$$ LRFD Eq 5.10.8-2

Bars Used ....# 5 @ 12 in $\quad \text{Area\_of\_Steel} := 0.31in^2 \quad As := \text{Area\_of\_Steel} = 0.31 \cdot in^2$

b is the least width of component section $\quad \text{comp\_width} := 12in \quad b := \text{comp\_width} = 12 \cdot in$

$h := \text{Deck\_Thickness} = 7 \cdot in$

$\text{yield\_strength} := 60ksi \qquad fy := \text{yield\_strength} = 60 \cdot ksi$

$$\text{Equation}_1 := \text{if}\left[As \ge \frac{1.30 \cdot b \cdot h \cdot kip}{2 \cdot (b+h) \cdot fy \cdot in}, \text{"YES"}, \text{"NO"}\right] \qquad \boxed{\text{Equation}_1 = \text{"YES"}}$$

$$\text{Equation}_2 : \quad 0.11in^2 \le \text{Area\_of\_Steel} \le 0.60in^2 \qquad \boxed{\text{Equation}_2 := \text{"YES"}}$$

Deck Reinforcement Summary

Main Reinforcement, Transverse (Bot.):	**Use #**	$bar_{main} := 5$	**bars at**	$Spa_{main} := 12 \cdot in$
Main Reinforcement, Transverse (Top):	**Use #**	$bar_{main} = 5$	**bars at**	$Spa_{main} = 12 \cdot in$
Distribution Reinforcement, Longitudinal (Bottom):	**Use #**	$bar_{DP} := 5$	**bars at**	$Spa_{DP} := 12 \cdot in$ (Max.)
Temperature and Shrinkage, Longitudinal (Top):	**Use #**	$bar_{TS} := 5$	**bars at**	$Spa_{TS} := 12 \cdot in$ (Max.)

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Appendix B: Empirical design method sample, 12-foot beam spacing

EMPIRICAL DECK DESIGN

Codes and Specification

- "AASHTO LRFD Bridge Design Specifications "American Association of State Highway and Transportation Officials (AASHTO), 6th Edition, 2012 with Interims through 2013.
- "Florida Department of Transportation Structures Design Guidelines for Load and Resistance Factor Design," January 2013 Edition.
- "Florida Department of Transportation Design Standards", 2014.

Unit Definitions:

$pcf := \frac{lbf}{ft^3}$ $\quad ksi := 1000psi$ $\quad kip := 1000 \cdot lbf$

Reinforcing Dimensions

Deck Design:

Bridge Length Definitions (SDG 4.2.1)

For establishing profilograph and deck thickness requirements, bridge structures are defined as Short Bridges or Long Bridges. The determining length is the length of the bridge structure measured along the Profile Grade Line (PGL) from front face of backwall at Begin Bridge to front face of backwall at End Bridge of the structure. Based upon this established length, the following definitions apply:

A. Short Bridges: Bridge structures less than or equal to 100 feet in PGL length.
B. Long Bridges: Bridge structures more than 100 feet in PGL length.

Input

(NOTE: For deck slabs, use same reinforcement Top & Bot. Do not include Integral Wearing Surface in "h" OR "cover")

Variables:

Beam Type: Beam := "FIB36" (Use 2, 3, 4, 5, 6, FBT72, FBT78, FIB36, FIB45, FIB54, FIB63, FIB72, FIB78)

Beam Spacing: $S_{beam} := 12ft + 0.0in$

Overhang: $Overhang := 4ft + 0.0in$

Begin Bridge Station: $Begin_{sta} := 0ft$

End Bridge Station: $End_{sta} := 70ft$

Skew Angle: $Skew_{angle} := 0 \cdot deg$

$PGL\_Length := (End_{sta} - Begin_{sta}) = 70\,ft$

Bridge_Def := if(PGL_Length ≤ 100ft, "Short Bridge" , "Long Bridge") Bridge_Def = "Short Bridge"

EMPIRICAL DECK DESIGN

Deck Thickness Determination:

T_Beams := "NO"   New_Const := "YES"

Deck_Thickness := if(T_Beams = "YES" ,6.5in, if(New_Const = "YES" ,if(Bridge_Def = "Short Bridge" ,9in,8.5in),"Check widening"))

Deck_Thickness = 9·in

$Deck_{DesignDepth}$:= Deck_Thickness

Beam Properties

Constants:

Top Flange Width: $b_{tf} = 48 \cdot in$

Web Width: $t_w = 7 \cdot in$

$Deck_{EffectiveLength} := (S_{beam} - t_w) - (b_{tf} - t_w) \cdot 0.5$ LRFD 9.7.2.3 for definition of effective length

$Deck_{EffectiveLength} = 9.708\ ft$

Deck Design   (SDG 4.2.4)

Empirical Design Method: For Category 1 structures meeting the criteria in LRFD [9.7.2.4] and are not subject to either staged construction or future widening, design deck slabs by the Empirical Design method of LRFD [9.7.2]. In lieu of the reinforcing requirements of LRFD [9.7.2.5], use no. 5 bars at 12-inch centers in both directions in both the top and bottom layers. Place two additional No. 5 bars between the primary transverse top slab bars (4-inch nominal spacing) in the slab overhangs to meet the TL-4 loading requirements for the FDOT standard traffic railings. Extend one of the additional bars to the midpoint between the exterior beam and the first interior beam; extend the second additional bar 36-inches beyond this midpoint. The maximum deck overhang is 6 feet, measured from the centerline of the exterior beam.

Skew Reinforcement: (SDG 4.2.11)   (LRFD 9.7.1.3)

Transverse_Skew_Reinf_Placement := if($Skew_{angle}$ < 15deg,"Parallel to skew entire slab length" ,"Prependicular_to_CL_span")

Transverse_Skew_Reinf_Placement = "Parallel to skew entire slab length"

Additional_Longitudinal_Reinf := if($Skew_{angle}$ < 15deg,"Not Needed" ," #5 @6" along span -both ends")

Additional_Longitudinal_Reinf = "Not Needed"

Additional_Trans_Reinf := if($Skew_{angle}$ < 15deg,"Not needed" ," Three #5 bars @6", full width,parallel to skew, top mat,each end ")

Additional_Trans_Reinf = "Not needed"

EMPIRICAL DECK DESIGN

Conditions to be satisfied to use the Empirical Design Method; LRFD [9.7.2]

0- Structure is category 1 as defined in PPM 26.3.1 $Condition_0 := "YES"$

1- Diaphragms used throughout the cross-section at line of support: $Condition_1 := "YES"$

2- Supporting Components are made of steel and/or concrete $Condition_2 := "YES"$

3- Deck is fully cast in place and water cured $Condition_3 := "YES"$

4- Deck is of uniform depth except for haunches at girder flanges and other local thickening: $Condition_4 := "YES"$

5- Ratio of effective length to design depth:

$Deck_{DesignDepth} = 9 \cdot in$

$$Eff\_Len\_to\_Design\_Thick\_Ratio := \frac{Deck_{EffectiveLength}}{Deck_{DesignDepth}}$$

$Eff\_Len\_to\_Design\_Thick\_Ratio = 12.944$

$Condition_5 := if(18 \geq Eff\_Len\_to\_Design\_Thick\_Ratio \geq 6, "YES", "NO")$ $Condition_5 = "YES"$

6- Slab core depth:

$Deck\_Thickness = 9 \cdot in$

$cover := 1.5in$

$slab\_core\_depth := Deck\_Thickness - 2 \cdot cover$ $slab\_core\_depth = 6 \cdot in$

$Condition_6 := if(slab\_core\_depth \geq 4in, "YES", "NO")$

$Condition_6 = "YES"$

7- Effective Length:

$Condition_7 := if\left(Deck_{EffectiveLength} \leq 13.5ft, "YES", "NO"\right)$

$Condition_7 = "YES"$

8- Minimum slab depth:

$Deck\_Thickness = 9 \cdot in$

$Condition_8 := if(Deck\_Thickness \geq 7in, "YES", "NO")$ $Condition_8 = "YES"$

EMPIRICAL DECK DESIGN

9- Overhang beyond the CL of the outer girder:

$Deck\_Thickness = 9 \cdot in$

$Condition_9 := if(Overhang \geq 3 \cdot Deck\_Thickness, "YES", "NO")$

(Structurally continuous concrete barrier is made composite with the overhang)

$Condition_9 = "YES"$

10- The specified 28-day strength of the concrete:

$f_c := 4.5ksi$

$Condition_{10} := if(f_c \geq 4ksi, "YES", "NO")$

$Condition_{10} = "YES"$

11 - The deck is made composite with the supporting structural components:

$Condition_{11} := "YES"$

Final Condition Check:

$j := 0..11$

$TotCheck_j := if(Condition_j = "YES", 0, 1)$

$TotalCheck := if(max(TotCheck) = 0, "OK", "No Good, one or more checks failed.")$

$TotalCheck = "OK"$

These are the failing sections if Total Check is not OK.

$if(TotalCheck = "OK", "All Good", match(1, TotCheck)) = "All Good"$

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EMPIRICAL DECK DESIGN

Temperature and Shrinkage Reinforcement **(SDG 4.2.12)**

For all cast in place decks, design temperature and shrinkage reinforcement per ***LRFD*** **[5.10.8]** except do not exceed 12 inch spacing and the minimum bar size is No. 4.

From LRFD 5.10.8:

For bars or welded wire fabric, the area of reinforcement per foot, on each face and in each direction, shall satisfy:

$$\text{Area\_of\_Steel} \geq \frac{1.30bh}{2(b+h)fy}$$ LRFD Eq 5.10.8-1

$$0.11in^2 \leq \text{Area\_of\_Steel} \leq 0.60in^2$$ LRFD Eq 5.10.8-2

Bars Used ....# 5 @ 12 in $\text{Area\_of\_Steel} := 0.31in^2$ $As := \text{Area\_of\_Steel} = 0.31 \cdot in^2$

b is the least width of component section $\text{comp\_width} := 12in$ $b := \text{comp\_width} = 12 \cdot in$

$h := \text{Deck\_Thickness} = 9 \cdot in$

$\text{yield\_strength} := 60ksi$ $fy := \text{yield\_strength} = 60 \cdot ksi$

$$\text{Equation}_1 := \text{if}\left[As \geq \frac{1.30 \cdot b \cdot h \cdot kip}{2 \cdot (b+h) \cdot fy \cdot in}, \text{"YES"}, \text{"NO"}\right]$$ $\text{Equation}_1 = \text{"YES"}$

Equation$_2$: $0.11in^2 \leq \text{Area\_of\_Steel} \leq 0.60in^2$ $\text{Equation}_2 := \text{"YES"}$

Deck Reinforcement Summary

Main Reinforcement, Transverse (Bot.):	**Use #**	$bar_{main} := 5$	**bars at**	$Spa_{main} := 12.in$
Main Reinforcement, Transverse (Top):	**Use #**	$bar_{main} = 5$	**bars at**	$Spa_{main} = 12 \cdot in$
Distribution Reinforcement, Longitudinal (Bottom):	**Use #**	$bar_{DP} := 5$	**bars at**	$Spa_{DP} := 12.in$ (Max.)
Temperature and Shrinkage, Longitudinal (Top):	**Use #**	$bar_{TS} := 5$	**bars at**	$Spa_{TS} := 12.in$ (Max.)

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Appendix C: Traditional design method sample, 6-foot beam spacing

LRFD TRADITIONAL DECK DESIGN (EQUIVALENT STRIP METHOD) **BEAM SPACING = 6 FEET**

Codes and Specification

- "AASHTO LRFD Bridge Design Specifications "American Association of State Highway and Transportation Officials (AASHTO), 6th Edition, 2012 with Interims through 2013.
- "Florida Department of Transportation Structures Design Guidelines for Load and Resistance Factor Design," January 2013 Edition.
- "Florida Department of Transportation Design Standards", 2014.

Reinforcing Dimensions

Input

(NOTE: For deck slabs, use same reinforcement Top & Bot. Do not include Integral Wearing Surface in "h" OR "cover")

Variables:

Beam Type: $Beam := \textbf{"FIB36"}$ (Use 2, 3, 4, 5, 6, FBT72, FBT78, FIB36, FIB45, FIB54, FIB63, FIB72, FIB78)

Concrete Strength: $f_c := \mathbf{5500}psi$ *(Assume Extremely Aggr Env., use Class IV CIP Bridge Deck Concrete, SDG Table 1.4.3-1)*

Concrete Weight: $w_c := 150 \cdot pcf$ *(SDG Table 2.2-1)*

Aggregate Correction Factor: $K_1 := 0.9$ *(SDG 1.4.1.A)*

Yield Strength of Reinforcing Steel: $f_y := 60ksi$

Width of Design Section: $b := 1ft$

Height of Design Section (deck thickness): $h := 8in$

(SDG 4.2.2.B - For new construction of "Short Bridges" other than Inverted-T Beam bridge superstructures, the minimum thickness of bridge decks cast-in-place (CIP) on beams or girders is 8-inches.

Height of Sacrificial Wearing Surface: $h_{sac} := 0.0in$

Load Reduction Factor for Moment (Initial Guess): $\phi_M := 0.90$

Top cover: $cover_t := 2in$ *(SDG Table 1.4.2-1)*

Bottom cover: $cover_b := 2in$ *(SDG Table 1.4.2-1)*

Exposure Condition: $exposure\_condition := \textbf{"Class 1"}$ *(SDG 4.1.8)*

Beam Spacing: $S_{beam} := \mathbf{6}ft + \mathbf{0.0}in$

Future Wearing Surface: $FWS := \mathbf{15}psf$ *(SDG Table 2.2-1)*

Weight of SIP Forms: $SIP := 20psf$

Width of Traffic Railing Barrier: $W_{barr} := 1.5ft$

Width of Raised Sidewalk: $W_{sw} := 0ft$

Traffic Railing Barrier: $w_{barrier} := 420plf$

Weight of Median: $w_{med} := \mathbf{0}ft \cdot b \cdot w_c$ $\qquad w_{med} = \mathbf{0} \cdot plf$

Bridge Skew: $Skew := mean[(\mathbf{0}),(\mathbf{0})]\,deg$ $\qquad Skew = \mathbf{0} \cdot deg$

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LRFD TRADITIONAL DECK DESIGN (EQUIVALENT STRIP METHOD) **BEAM SPACING = 6 FEET**

Weight of Sidewalk: $w_{sw} := \boxed{0in + (W_{barr} + W_{sw}) \cdot 0.00} \cdot (W_{barr} + W_{sw}) \cdot w_c$ $w_{sw} = \mathbf{0} \cdot plf$

Weight of Pedestrian/Bicycle Railing and Fence: $w_{barr\_ped} := 0plf + 0plf$ $w_{barr\_ped} = \mathbf{0} \cdot plf$

Beam Properties

Constants:

Top Flange Width: $b_{tf} = \mathbf{48} \cdot in$

Web Width: $t_w = \mathbf{7} \cdot in$

Concrete Unit Weight for Modulus of Elasticity Calc: $\gamma_c := \mathbf{145} pcf$ *(SDG 1.4.1.A)*

Modulus of Elasticity Deck: $E_c := \mathbf{33000} \cdot K_1 \cdot \left(\frac{\gamma_c}{kcf}\right)^{\mathbf{1.5}} \cdot \sqrt{\frac{f_c}{ksi}} \cdot ksi$ $E_c = \mathbf{3845.8} \cdot ksi$ *(AASHTO 5.4.2.4)*

Modulus of Elasticity Reinforcing Steel: $E_s := \mathbf{29000} ksi$ *(AASHTO 5.4.3.2)*

Modular Ratio: $n := round\left(\frac{E_s}{E_c}\right)$ $n = \mathbf{8}$ *(AASHTO 5.7.1)*

Area of Deck Section: $A_c := h \cdot b$ $A_c = \mathbf{96} \cdot in^2$

Crack Control Exposure Condition Factor: $\gamma_e := \begin{cases} \mathbf{1.00} & \text{if exposure\_condition} = \text{"Class 1"} \\ \mathbf{0.75} & \text{otherwise} \end{cases}$ $\gamma_e = \mathbf{1}$ *(AASHTO 5.7.3.4)*

Design Moment

Location of Negative Live Load Design Moment:

AASHTO LRFD 4.6.2.1.6 --> The design section for negative moments and shear forces, where investigated, may be taken as follows:

For precast I-shaped concrete beams ....crossection (k) from Table 4.6.2.2.1-1: one-third the flange width, but not exceeding 15.0 in from the centerline of support.

The negative live load design moment is taken at a distance from the supports: $Loc_{negative} := \min\left(\frac{1}{3} \cdot b_{tf}, \mathbf{15} \cdot in\right)$ $Loc_{negative} = \mathbf{15.0} \cdot in$

AASHTO Table A4-1 - Deck Slab Design Table

Dead Load Moments for Moment Analysis:

"DC" loads include the dead load of structural components and non-structural attachments

Self-weight of Deck Slab: $w_{slab} := \left[(h + h_{sac}) \cdot b\right] \cdot w_c$ $w_{slab} = \mathbf{0.100} \cdot klf$

Weight of Traffic Railing Barriers: $P_{barrier} := w_{barrier} \cdot b$ $P_{barrier} = \mathbf{0.420} \cdot kip$

LRFD TRADITIONAL DECK DESIGN (EQUIVALENT STRIP METHOD) **BEAM SPACING = 6 FEET**

Weight of Pedestrian/Bicycle Railing and Fence: $P_{barTypeK} := w_{barr\_ped} \cdot b$ $P_{barTypeK} = \mathbf{0.000} \cdot kip$

Weight of Median: $w_{median} := w_{med}$ $w_{median} = \mathbf{0.000} \cdot klf$

Weight of Sidewalk: $w_{sw} := \dfrac{w_{sw} \cdot b}{W_{sw} + W_{barr}}$ $w_{sw} = \mathbf{0.000} \cdot klf$

Stay-in-Place Forms: $w_{sip} := SIP \cdot b$ $w_{sip} = \mathbf{0.020} \cdot klf$

"DW" loads include the dead load of a future wearing surface and utilities

Weight of Future Wearing Surface: $w_{fws} := FWS \cdot b$ $w_{fws} = \mathbf{0.015} \cdot klf$

Max. Positive Live Load Moment:

From AASHTO Table A4-1: $M_{LL\_pos} := M_{LL\_pos} \cdot kip \cdot \dfrac{ft}{ft}$ $M_{LL\_pos} = \mathbf{4.83} \cdot kip \cdot \dfrac{ft}{ft}$

Max. Negative Live Load Moment:

From AASHTO Table A4-1 for interpolation:

Minimum Distance Negative Moment and Location:

$Min_{neg\_loc} := Min_{neg\_loc} \cdot in$ $Min_{neg\_loc} = \mathbf{12} \cdot in$

$Min_{LL\_neg} := Min_{LL\_neg} \cdot kip \cdot \dfrac{ft}{ft}$ $Min_{LL\_neg} = \mathbf{2.31} \cdot kip \cdot \dfrac{ft}{ft}$

Maximum Distance Negative Moment and Location:

$Max_{neg\_loc} := Max_{neg\_loc} \cdot in$ $Max_{neg\_loc} = \mathbf{18} \cdot in$

$Max_{LL\_neg} := Max_{LL\_neg} \cdot kip \cdot \dfrac{ft}{ft}$ $Max_{LL\_neg} = \mathbf{1.39} \cdot kip \cdot \dfrac{ft}{ft}$

From AASHTO Table A4-1: (by interpolation)

$$M_{LL\_neg} := \left(Loc_{negative} - Min_{neg\_loc}\right) \cdot \left[\frac{\left(Max_{LL\_neg} - Min_{LL\_neg}\right)}{\left(Max_{neg\_loc} - Min_{neg\_loc}\right)}\right] + Min_{LL\_neg}$$

$M_{LL\_neg} = \mathbf{1.85} \cdot kip \cdot \dfrac{ft}{ft}$

LRFD TRADITIONAL DECK DESIGN (EQUIVALENT STRIP METHOD) **BEAM SPACING = 6 FEET**

Summary of Moments:

Max Moments between the beams:

Max. Positive Service DC Moment:	$M_{DC\_Pos} := \mathbf{4.27} \cdot kip \cdot \frac{in}{ft}$	$M_{DC\_Pos} = \mathbf{0.356} \cdot kip \cdot \frac{ft}{ft}$	(See STAAD Output)
Max. Negitive Service DC Moment:	$M_{DC\_Neg} := \mathbf{5.933} \cdot kip \cdot \frac{in}{ft}$	$M_{DC\_Neg} = \mathbf{0.494} \cdot kip \cdot \frac{ft}{ft}$	(See STAAD Output)
Max. Positive Service DW Moment:	$M_{DW\_Pos} := \mathbf{0.355} \cdot kip \cdot \frac{in}{ft}$	$M_{DW\_Pos} = \mathbf{0.03} \cdot kip \cdot \frac{ft}{ft}$	(See STAAD Output)
Max. Negitive Service DW Moment:	$M_{DW\_Neg} := \mathbf{0.605} \cdot kip \cdot \frac{in}{ft}$	$M_{DW\_Neg} = \mathbf{0.05} \cdot kip \cdot \frac{ft}{ft}$	(See STAAD Output)

Load Combinations:

Maximum Service I Moment:

$$M_{Service} := \max\left(M_{DC\_Pos} + M_{DW\_Pos} + M_{LL\_pos}, M_{DC\_Neg} + M_{DW\_Neg} + M_{LL\_neg}\right) \cdot ft$$

$$M_{Service} = \mathbf{5.215} \cdot kip \cdot ft$$

Maximum Strength I Moment:

$$M_{Strength} := \max\begin{pmatrix} 1.25 M_{DC\_Pos} + \mathbf{1.50} \cdot M_{DW\_Pos} \cdots, 1.25 M_{DC\_Neg} + \mathbf{1.5} \cdot M_{DW\_Neg} \cdots \\ + 1.75 M_{LL\_pos} \qquad\qquad + 1.75 M_{LL\_neg} \end{pmatrix} \cdot ft$$

$$M_{Strength} = \mathbf{8.942} \cdot kip \cdot ft$$

Applied Moment: $M_{applied} := M_{Strength}$ $\qquad M_{applied} = \mathbf{8.942} \cdot kip \cdot ft$

Flexure Reinforcement

Minimum Reinforcement: *(AASHTO 5.7.3.3.2 & See AASHTO 5.7.2 for Design Assumptions)*

Modulus of Rapture: *(AASHTO 5.4.2.6 & SDG 1.4.1.B)* $\quad f_r := \mathbf{0.24} \cdot \sqrt{\frac{f_c}{ksi}} \cdot ksi \qquad f_r = \mathbf{562.85} \cdot psi$

Moment of Intertia of Slab Section: $\quad I_g := \frac{b \cdot h^{\mathbf{3}}}{\mathbf{12}} \qquad I_g = \mathbf{512} \cdot in^{\mathbf{4}}$

Distance from the Extreme Tensile Fiber to the Neutral Axis of the Composite Section: $\quad y_t := \frac{h}{\mathbf{2}} \qquad y_t = \mathbf{4} \cdot in$

Cracking Moment: $\quad M_{cr} := \frac{f_r \cdot I_g}{y_t} \qquad M_{cr} = \mathbf{6.004} \cdot kip \cdot ft$

LRFD TRADITIONAL DECK DESIGN (EQUIVALENT STRIP METHOD) **BEAM SPACING = 6 FEET**

Cracking Moment Limit: $1.2 \cdot M_{cr} = 7.204 \cdot kip \cdot ft$

Desgin Ultimate Moment: $M_u := \begin{cases} M_{Strength} & \text{if } M_{Strength} \geq 1.2M_{cr} \\ \min(1.33M_{Strength}, 1.2M_{cr}) & \text{otherwise} \end{cases}$ $M_u = 8.942 \cdot kip \cdot ft$

Distance from Extreme Compressive Fiber to Centroid of Reinforcing Steel: $d_e := h - cover_b - \frac{d_{bar}(bar_{main})}{2}$ $d_e = 5.688 \cdot in$

Nominal Strength Coefficient of Resistance: $R_u := \frac{M_u}{\phi_M \cdot b \cdot d_e^2}$ $R_u = 307.138 \cdot psi$

$m := \frac{f_y}{0.85 \cdot f_c}$ $m = 12.834$

ACI ρ Equation: $\rho := \frac{1}{m} \cdot \left(1 - \sqrt{1 - \frac{2 \cdot \frac{R_u}{psi} \cdot m}{\frac{f_y}{psi}}}\right)$ $\rho = 0.005299$

$A_{sreqdpos} := \rho \cdot b \cdot d_e$ $A_{sreqdpos} = 0.362 \cdot in^2$

Minimum Required A_s between beams $A_{sMinReq} := A_{sreqdpos}$ $A_{sMinReq} = 0.362 \cdot in^2$

Use Main Reinforcing: $bar_{main} \equiv 5$ $Spa_{main} := 9.5in$

Diameter of Bar: $d_b := d_{bar}(bar_{main})$ $d_b = 0.625 \cdot in$

Area of Reinforcing in a Section 1 ft Wide: $A\_s(z, cc) := A_{bar}(z) \cdot \frac{12 \cdot in}{cc}$

Area of Reinforcing: $A_{sMain} := A\_s(bar_{main}, Spa_{main})$ $A_{sMain} = 0.392 \cdot in^2$

Depth of Equivalent Rectangular Stress Block: $a := \frac{A_{sMain} \cdot f_y}{0.85 \cdot f_c \cdot b}$ $a = 0.419 \cdot in$

Ratio of Reinforcement Provided: $\rho := \frac{A_{sMain}}{b \cdot d_e}$ $\rho = 0.0057$

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LRFD TRADITIONAL DECK DESIGN (EQUIVALENT STRIP METHOD) **BEAM SPACING = 6 FEET**

Determine ϕ (Tension or Compression Controlled Section): *(AASHTO 5.7.2.1 & AASHTO 5.7.2.2)*

Determine location of N.A using Whitney Stress Block.:

$$\beta_1 := \begin{cases} \max\left[\left[0.85 - 0.05\cdot\left(\frac{f_c - 4\cdot ksi}{ksi}\right)\right], 0.65\right] & \text{if } f_c > 4ksi \\ 0.85 & \text{otherwise} \end{cases} \qquad \beta_1 = 0.775$$

Distance from the Extreme Compression Fiber to the N.A.:

$$c_{comp} := \frac{a}{\beta_1} \qquad c_{comp} = 0.54\cdot in$$

Actual Tensile Strain in Extreme Tension Steel:

$$\varepsilon_T := 0.003\cdot\frac{d_e - c_{comp}}{c_{comp}} \qquad \text{(AASHTO Figure C5.7.2.1-1)} \qquad \varepsilon_T = 0.029$$

Comp. and Tension Controlled Section Limits of Net Tensile Strain in the Extreme Tension Steel:

$$\varepsilon_{T\_Limits} := \begin{pmatrix} 0.002 \\ 0.005 \end{pmatrix}$$

Compresssion Controlled if ε_T <or= 0.0002

Tension Controlled if ε_T >or= 0.0005

Comp. and Tension Controlled Reinforced Concrete Section Resistance Factors:

$$\phi := \begin{pmatrix} 0.75 \\ 0.9 \end{pmatrix}$$

Compresssion Controlled

Tension Controlled

(AASHTO 5.5.4.2.1)

Determine Controlling Force:

$$\text{Controlling} := \begin{cases} \text{"Compression"} & \text{if } \varepsilon_T \le \varepsilon_{T\_Limits_0} \\ \text{"Tension"} & \text{if } \varepsilon_T \ge \varepsilon_{T\_Limits_1} \\ \text{"In Transition"} & \text{otherwise} \end{cases} \qquad \text{Controlling} = \text{"Tension"}$$

Determine Controlling Resistance Factor:

$$\phi_M := \begin{cases} \phi_0 & \text{if Controlling} = \text{"Compression"} \\ \phi_1 & \text{if Controlling} = \text{"Tension"} \\ \text{linterp}(\varepsilon_{T\_Limits}, \phi, \varepsilon_T) & \text{otherwise} \end{cases} \qquad \phi_M = 0.9$$

Factored Flexural Resistance:

$$\phi M_n := \phi_M\cdot A_{sMain}\cdot f_y\cdot\left(d_e - \frac{a}{2}\right) \qquad \phi M_n = 9.653\cdot kip\cdot ft$$

Ultimate Moment:

$$M_u = 8.942\cdot kip\cdot ft$$

Check Moment Capacity:

$$\text{CheckMoment} := \begin{cases} \text{"OK"} & \text{if } \phi M_n \ge M_u \\ \text{"No Good"} & \text{if } \phi M_n < M_u \end{cases} \qquad \boxed{\text{CheckMoment} = \text{"OK"}}$$

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LRFD TRADITIONAL DECK DESIGN (EQUIVALENT STRIP METHOD) **BEAM SPACING = 6 FEET**

Crack Control Check

(AASHTO 5.7.3.4)

Thickness of Concrete Cover Measured from Extreme Tension Fiber to Center of the Flexural Reinforcement Located Closest Thereto: $d_c := cover_b + \frac{d_b}{2}$ $d_c = 2.313 \cdot in$

Depth of Neutral Axis $f(x) := \frac{b \cdot x^2}{2 \cdot A_{sMain} \cdot n} + x - d_e$

$x := root(f(x), x, 0, d_e)$ $x = 1.482 \cdot in$

Tensile Stress in Reinforcement at the Service Limit State: $f_{ss} := \frac{M_{Service}}{A_{sMain} \cdot \left(d_e - \frac{x}{3}\right)}$ $f_{ss} = 30.774 \cdot ksi$

Ratio of Flexural Strain at the Extreme Tension Face to the Strain at the Centroid of the Reinforcement Layer Nearest the Tension Face: $\beta_s := 1 + \frac{d_c}{0.7 \cdot (h - d_c)}$ *(AASHTO 5.7.3.4-1)* $\beta_s = 1.581$

Maximum Reinforcement Spacing for Crack Control: $s_{max} := \left(\frac{700 \cdot \gamma_e}{\beta_s \cdot \frac{f_{ss}}{ksi}} - 2 \cdot \frac{d_c}{in}\right) \cdot in$ *(AASHTO 5.7.3.4-1)* $s_{max} = 9.764 \cdot in$

Reiforcement Spacing Provided: $s_{actual} := Spa_{main}$ $s_{actual} = 9.5 \cdot in$

Check Spacing: $CheckSpacing := \begin{vmatrix} \text{"No Good"} & \text{if } s_{actual} > s_{max} \\ \text{"OK"} & \text{otherwise} \end{vmatrix}$ $CheckSpacing = \text{"OK"}$

Distribution Reinforcement

For primary reinforcement perpendicular to traffic *(AASHTO 9.7.3.2)*

Distance Between Beam Flange Tips: $D1 := S_{beam} - b_{tf}$ $D1 = 2\,ft$

Flange Overhang: $D2 := b_{tf} - t_w$ $D2 = 3.417\,ft$

Effective Span Length: $sp_{eff} := D1 + D2$ *(AASHTO 9.7.2.3)* $sp_{eff} = 5.417\,ft$

Distribution Reinforcement %: $DR := \min\left(\frac{220}{\sqrt{\frac{sp_{eff}}{ft}}}, 67\right) \cdot \%$ $DR = 67 \cdot \%$

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LRFD TRADITIONAL DECK DESIGN (EQUIVALENT STRIP METHOD) **BEAM SPACING = 6 FEET**

Area of Steel Required: $A_{sDR} := DR \cdot A_{sMain}$ $A_{sDR} = 0.262 \cdot in^2$

Use Dist. Reinforcement: $bar_{DP} := 5$ $Spa_{DP} := 12in$ (Max.)

Area of Reinforcing Provided: $A_{sDP} := A\_s(bar_{DP}, Spa_{DP})$ $A_{sDP} = 0.31 \cdot in^2$

$$CheckDistRein := \begin{cases} \text{"OK"} & \text{if } A_{sDP} \geq A_{sDR} \\ \text{"No Good"} & \text{otherwise} \end{cases}$$

$CheckDistRein = \text{"OK"}$

Temperature & Shrinkage Reinforcement:

(AASHTO 5.10.8.2)

Area of Steel Required for Temp & Shrinkage:

$$A_{s\_ts} := \begin{cases} 0.11 \frac{in^2}{ft} & \text{if } \dfrac{1.30 \cdot \frac{kip}{in \cdot ft} \cdot b \cdot h}{2 \cdot (b+h) \cdot f_y} \leq 0.11 \frac{in^2}{ft} \\ 0.60 \frac{in^2}{ft} & \text{if } \dfrac{1.30 \cdot \frac{kip}{in \cdot ft} \cdot b \cdot h}{2 \cdot (b+h) \cdot f_y} \geq 0.60 \frac{in^2}{ft} \\ \dfrac{1.30 \cdot \frac{kip}{in \cdot ft} \cdot b \cdot h}{2 \cdot (b+h) \cdot f_y} & \text{otherwise} \end{cases}$$

$A_{s\_ts} = 0.11 \cdot \frac{in^2}{ft}$

Use: $bar_{TS} := 5$ *(SDG 4.2.11 - #4 Min.)* $Spa_{TS} := 12in$ *(SDG 4.2.11 - 12in Max.)*

Area of Temp and Shrink Reinforcing: $A_{sTS} := A\_s(bar_{TS}, Spa_{TS})$ $A_{sTS} = 0.31 \cdot in^2$

$$Check_{TS} := \begin{cases} \text{"OK"} & \text{if } A_{sDR} \geq A_{s\_ts} \cdot ft \wedge A_{sMain} \geq A_{s\_ts} \cdot ft \wedge A_{sTS} \geq A_{s\_ts} \cdot ft \\ \text{"No Good"} & \text{otherwise} \end{cases}$$

$Check_{TS} = \text{"OK"}$

Maximum Spacing of Temperature and Shrinkage Reinforcement: $s_{TSmax} := \min(3 \cdot h, 18in, 12in)$ $s_{TSmax} = 12 \cdot in$

$$Check_{Spa} := \begin{cases} \text{"OK"} & \text{if } Spa_{DP} \leq s_{TSmax} \wedge Spa_{main} \leq s_{TSmax} \wedge Spa_{TS} \leq s_{TSmax} \\ \text{"No Good"} & \text{otherwise} \end{cases}$$

$Check_{Spa} = \text{"OK"}$

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LRFD TRADITIONAL DECK DESIGN (EQUIVALENT STRIP METHOD) **BEAM SPACING = 6 FEET**

Deck Reinforcement Summary

Main Reinforcement, Transverse (Bot.):	**Use #**	$bar_{main} = \mathbf{5}$	**bars at**	$Spa_{main} = \mathbf{9.5} \cdot in$
Main Reinforcement, Transverse (Top):	**Use #**	$bar_{main} = \mathbf{5}$	**bars at**	$Spa_{main} = \mathbf{9.5} \cdot in$
Distribution Reinforcement, Longitudinal (Bottom):	**Use #**	$bar_{DP} = \mathbf{5}$	**bars at**	$Spa_{DP} = \mathbf{12} \cdot in$ (Max.)
Temperature and Shrinkage, Longitudinal (Top):	**Use #**	$bar_{TS} = \mathbf{5}$	**bars at**	$Spa_{TS} = \mathbf{12} \cdot in$ (Max.)

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	Part Deck DC & DW Loads		
Job Title Georges El-Gharib Thesis	Ref		
	By GIE	Date 15-Sept-13	Chd
Client UNF	File 6 foot DC Moments.std	Date/Time 16-Sep-2013 20:13	

Job Information

	Engineer	Checked	Approved
Name:	GIE		
Date:	15-Sept-13		

Comments

Finding Dead Load Moments for deck design

Structure Type	PLANE FRAME

Number of Nodes	9	Highest Node	9
Number of Elements	8	Highest Beam	8

Number of Basic Load Cases	2
Number of Combination Load Cases	0

Included in this printout are data for:

All	The Whole Structure

Included in this printout are results for load cases:

Type	L/C	Name
Primary	1	DC COMPONENTS DEAD LOADS
Primary	2	DW FWS DEAD LOAD

Nodes

Node	X (ft)	Y (ft)	Z (ft)
1	0.000	0.000	0.000
2	4.000	0.000	0.000
3	10.000	0.000	0.000
4	16.000	0.000	0.000
5	22.000	0.000	0.000
6	28.000	0.000	0.000
7	34.000	0.000	0.000
8	40.000	0.000	0.000
9	44.000	0.000	0.000

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	Part: Deck DC & DW Loads		
Job Title: Georges El-Gharib Thesis	Ref		
	By: GIE	Date: 15-Sept-13	Chd
Client: UNF	File: 6 foot DC Moments.std	Date/Time: 16-Sep-2013 20:13	

Beams

Beam	Node A	Node B	Length (ft)	Property	β (degrees)
1	2	1	4.000	1	0
2	2	3	6.000	1	0
3	3	4	6.000	1	0
4	4	5	6.000	1	0
5	5	6	6.000	1	0
6	6	7	6.000	1	0
7	7	8	6.000	1	0
8	8	9	4.000	1	0

Materials

Mat	Name	E (kip/in^2)	ν	Density (kip/in^3)	α (/°F)
1	STEEL	29E+3	0.300	0.000	6E -6
2	STAINLESSSTEEL	28E+3	0.300	0.000	10E -6
3	ALUMINUM	10E+3	0.330	0.000	13E -6
4	CONCRETE	3.15E+3	0.170	0.000	5E -6

Supports

Node	X (kip/in)	Y (kip/in)	Z (kip/in)	rX (kip ft/deg)	rY (kip ft/deg)	rZ (kip ft/deg)
2	Fixed	Fixed	Fixed	-	-	-
3	-	Fixed	-	-	-	-
4	-	Fixed	-	-	-	-
5	-	Fixed	-	-	-	-
6	-	Fixed	-	-	-	-
7	-	Fixed	-	-	-	-
8	Fixed	Fixed	Fixed	-	-	-

Basic Load Cases

Number	Name
1	DC COMPONENTS DEAD LOADS
2	DW FWS DEAD LOAD

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Beam Maximum Moments

Distances to maxima are given from beam end A.

Beam	Node A	Length (ft)	L/C		d (ft)	Max My (kip`in)	d (ft)	Max Mz (kip`in)
1	2	4.000	1:DC COMPON	Max -ve	0.000	0.000	0.000	26.640
				Max +ve	0.000	0.000	4.000	-0.000
			2:DW FWS DE	Max -ve	0.000	0.000	0.000	1.440
				Max +ve	0.000	0.000	4.000	-0.000
2	2	6.000	1:DC COMPON	Max -ve	0.000	0.000	0.000	26.640
				Max +ve	0.000	0.000	6.000	-1.512
			2:DW FWS DE	Max -ve	0.000	0.000	0.000	1.440
				Max +ve	0.000	0.000	4.000	-0.037
3	3	6.000	1:DC COMPON	Max -ve	0.000	0.000	6.000	5.933
				Max +ve	0.000	0.000	2.000	-4.790
			2:DW FWS DE	Max -ve	0.000	0.000	6.000	0.605
				Max +ve	0.000	0.000	2.500	-0.358
4	4	6.000	1:DC COMPON	Max -ve	0.000	0.000	0.000	5.933
				Max +ve	0.000	0.000	3.500	-1.767
			2:DW FWS DE	Max -ve	0.000	0.000	0.000	0.605
				Max +ve	0.000	0.000	3.000	-0.253
5	5	6.000	1:DC COMPON	Max -ve	0.000	0.000	6.000	5.933
				Max +ve	0.000	0.000	2.500	-1.767
			2:DW FWS DE	Max -ve	0.000	0.000	6.000	0.605
				Max +ve	0.000	0.000	3.000	-0.253
6	6	6.000	1:DC COMPON	Max -ve	0.000	0.000	0.000	5.933
				Max +ve	0.000	0.000	4.000	-4.790
			2:DW FWS DE	Max -ve	0.000	0.000	0.000	0.605
				Max +ve	0.000	0.000	3.500	-0.358
7	7	6.000	1:DC COMPON	Max -ve	0.000	0.000	6.000	26.640
				Max +ve	0.000	0.000	0.000	-1.512
			2:DW FWS DE	Max -ve	0.000	0.000	6.000	1.440
				Max +ve	0.000	0.000	2.000	-0.037
8	8	4.000	1:DC COMPON	Max -ve	0.000	0.000	0.000	26.640
				Max +ve	0.000	0.000		
			2:DW FWS DE	Max -ve	0.000	0.000	0.000	1.440
				Max +ve	0.000	0.000		

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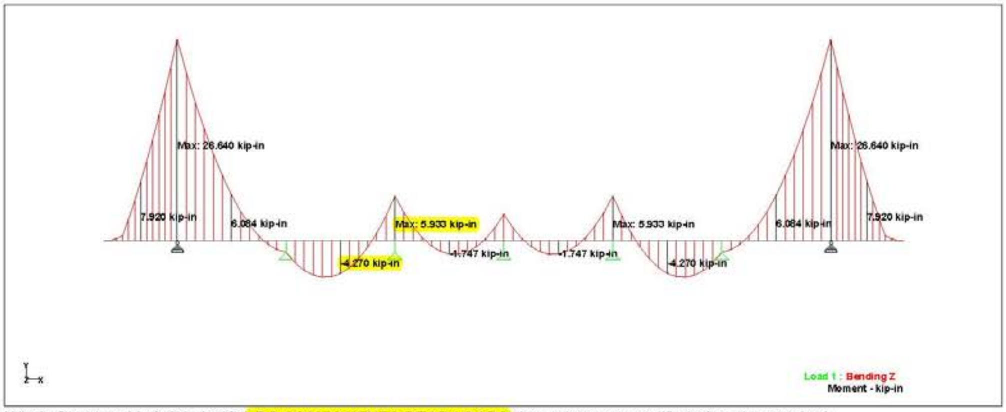
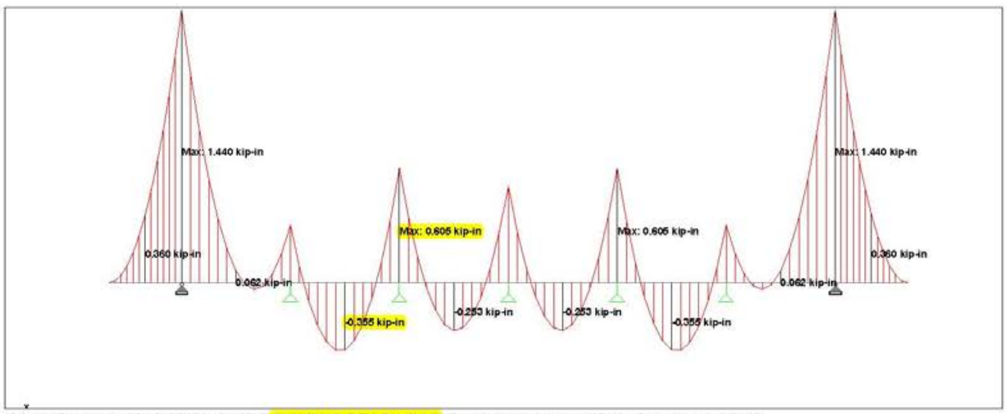
	By GIE	Date 15-Sept-13	Chd
Client UNF	File 6 foot DC Moments.std	Date/Time 16-Sep-2013 20:13	



Whole Structure Mz 2.4kip-in:1ft 1 DC COMPONENTS DEAD LOADS (Input data was modified after picture taken)



Whole Structure Mz 0.096kip-in:1ft 2 DW FWS DEAD LOAD (Input data was modified after picture taken)

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Appendix D: Traditional design method sample, 12-foot beam spacing

LRFD TRADITIONAL DECK DESIGN (EQUIVALENT STRIP METHOD) **BEAM SPACING = 12 FEET**

Codes and Specification

- "AASHTO LRFD Bridge Design Specifications "American Association of State Highway and Transportation Officials (AASHTO), 6th Edition, 2012 with Interims through 2013.
- "Florida Department of Transportation Structures Design Guidelines for Load and Resistance Factor Design," January 2013 Edition.
- "Florida Department of Transportation Design Standards", 2014.

Reinforcing Dimensions

Input

(NOTE: For deck slabs, use same reinforcement Top & Bot. Do not include Integral Wearing Surface in "h" OR "cover")

Variables:

Beam Type:	Beam := **"FIB36"**	(Use 2, 3, 4, 5, 6, FBT72, FBT78, FIB36, FIB45, FIB54, FIB63, FIB72, FIB78)
Concrete Strength:	$f_c := \mathbf{5500} psi$	*(Assume Extremely Aggr Env., use Class IV CIP Bridge Deck Concrete, SDG Table 1.4.3-1)*
Concrete Weight:	$w_c := 150 \cdot pcf$	*(SDG Table 2.2-1)*
Aggregate Correction Factor:	$K_1 := 0.9$	*(SDG 1.4.1.A)*
Yield Strength of Reinforcing Steel:	$f_y := 60ksi$	
Width of Design Section:	$b := 1ft$	
Height of Design Section (deck thickness):	$h := 8in$	*(SDG 4.2.2.B - For new construction of "Short Bridges" other than Inverted-T Beam bridge superstructures, the minimum thickness of bridge decks cast-in-place (CIP) on beams or girders is 8-inches.*
Height of Sacrificial Wearing Surface:	$h_{sac} := 0.0in$	
Load Reduction Factor for Moment (Initial Guess):	$\phi_M := 0.90$	
Top cover:	$cover_t := 2in$	*(SDG Table 1.4.2-1)*
Bottom cover:	$cover_b := 2in$	*(SDG Table 1.4.2-1)*
Exposure Condition:	exposure_condition := **"Class 1"**	*(SDG 4.1.8)*
Beam Spacing:	$S_{beam} := \mathbf{12}ft + \mathbf{0.0}in$	
Future Wearing Surface:	$FWS := \mathbf{15}psf$	*(SDG Table 2.2-1)*
Weight of SIP Forms:	$SIP := 20psf$	
Width of Traffic Railing Barrier:	$W_{barr} := 1.5ft$	
Width of Raised Sidewalk:	$W_{sw} := 0ft$	
Traffic Railing Barrier:	$w_{barrier} := 420plf$	
Weight of Median:	$w_{med} := \mathbf{0}ft \cdot b \cdot w_c$	$w_{med} = \mathbf{0} \cdot plf$
Bridge Skew:	$Skew := mean[(\mathbf{0}),(\mathbf{0})] deg$	$Skew = \mathbf{0} \cdot deg$

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LRFD TRADITIONAL DECK DESIGN (EQUIVALENT STRIP METHOD) **BEAM SPACING = 12 FEET**

Weight of Sidewalk:	$w_{sw} := \left[0in + (W_{barr} + W_{sw})\cdot 0.00\right]\cdot(W_{barr} + W_{sw})\cdot w_c$	$w_{sw} = \mathbf{0}\cdot plf$	
Weight of Pedestrian/Bicycle Railing and Fence:	$w_{barr\_ped} := 0plf + 0plf$	$w_{barr\_ped} = \mathbf{0}\cdot plf$	

▸ Beam Properties

Constants:

Top Flange Width:	$b_{tf} = \mathbf{48}\cdot in$		
Web Width:	$t_w = \mathbf{7}\cdot in$		
Concrete Unit Weight for Modulus of Elasticity Calc:	$\gamma_c := \mathbf{145}pcf$		*(SDG 1.4.1.A)*
Modulus of Elasticity Deck:	$E_c := \mathbf{33000}\cdot K_1\cdot\left(\frac{\gamma_c}{kcf}\right)^{\mathbf{1.5}}\cdot\sqrt{\frac{f_c}{ksi}}\cdot ksi$	$E_c = \mathbf{3845.8}\cdot ksi$	*(AASHTO 5.4.2.4)*
Modulus of Elasticity Reinforcing Steel:	$E_s := \mathbf{29000}ksi$		*(AASHTO 5.4.3.2)*
Modular Ratio:	$n := round\left(\frac{E_s}{E_c}\right)$	$n = \mathbf{8}$	*(AASHTO 5.7.1)*
Area of Deck Section:	$A_c := h\cdot b$	$A_c = \mathbf{96}\cdot in^{\mathbf{2}}$	
Crack Control Exposure Condition Factor:	$\gamma_e := \begin{cases} \mathbf{1.00} & \text{if exposure\_condition} = \text{"Class 1"} \\ \mathbf{0.75} & \text{otherwise} \end{cases}$	$\gamma_e = \mathbf{1}$	*(AASHTO 5.7.3.4)*

Design Moment

Location of Negative Live Load Design Moment:

AASHTO LRFD 4.6.2.1.6 --> The design section for negative moments and shear forces, where investigated, may be taken as follows:

For precast I-shaped concrete beams ....crossection (k) from Table 4.6.2.2.1-1: one-third the flange width, but not exceeding 15.0 in from the centerline of support.

The negative live load design moment is taken at a distance from the supports: $Loc_{negative} := \min\left(\frac{\mathbf{1}}{\mathbf{3}}\cdot b_{tf}, \mathbf{15}\cdot in\right)$ $Loc_{negative} = \mathbf{15.0}\cdot in$

▸ AASHTO Table A4-1 - Deck Slab Design Table

Dead Load Moments for Moment Analysis:

"DC" loads include the dead load of structural components and non-structural attachments

Self-weight of Deck Slab:	$w_{slab} := \left[(h + h_{sac})\cdot b\right]\cdot w_c$	$w_{slab} = \mathbf{0.100}\cdot klf$
Weight of Traffic Railing Barriers:	$P_{barrier} := w_{barrier}\cdot b$	$P_{barrier} = \mathbf{0.420}\cdot kip$

LRFD TRADITIONAL DECK DESIGN (EQUIVALENT STRIP METHOD) **BEAM SPACING = 12 FEET**

Weight of Pedestrian/Bicycle Railing and Fence: $P_{barTypeK} := w_{barr\_ped} \cdot b$ $P_{barTypeK} = \mathbf{0.000} \cdot kip$

Weight of Median: $w_{median} := w_{med}$ $w_{median} = \mathbf{0.000} \cdot klf$

Weight of Sidewalk: $w_{sw} := \dfrac{w_{sw} \cdot b}{W_{sw} + W_{barr}}$ $w_{sw} = \mathbf{0.000} \cdot klf$

Stay-in-Place Forms: $w_{sip} := SIP \cdot b$ $w_{sip} = \mathbf{0.020} \cdot klf$

"DW" loads include the dead load of a future wearing surface and utilities

Weight of Future Wearing Surface: $w_{fws} := FWS \cdot b$ $w_{fws} = \mathbf{0.015} \cdot klf$

Max. Positive Live Load Moment:

From AASHTO Table A4-1: $M_{LL\_pos} := M_{LL\_pos} \cdot kip \cdot \dfrac{ft}{ft}$ $M_{LL\_pos} = \mathbf{8.01} \cdot kip \cdot \dfrac{ft}{ft}$

Max. Negative Live Load Moment:

From AASHTO Table A4-1 for interpolation:

Minimum Distance Negative Moment and Location:

$Min_{neg\_loc} := Min_{neg\_loc} \cdot in$ $Min_{neg\_loc} = \mathbf{12} \cdot in$

$Min_{LL\_neg} := Min_{LL\_neg} \cdot kip \cdot \dfrac{ft}{ft}$ $Min_{LL\_neg} = \mathbf{6.74} \cdot kip \cdot \dfrac{ft}{ft}$

Maximum Distance Negative Moment and Location:

$Max_{neg\_loc} := Max_{neg\_loc} \cdot in$ $Max_{neg\_loc} = \mathbf{18} \cdot in$

$Max_{LL\_neg} := Max_{LL\_neg} \cdot kip \cdot \dfrac{ft}{ft}$ $Max_{LL\_neg} = \mathbf{5.56} \cdot kip \cdot \dfrac{ft}{ft}$

From AASHTO Table A4-1: (by interpolation)

$$M_{LL\_neg} := \left(Loc_{negative} - Min_{neg\_loc}\right) \cdot \left[\frac{\left(Max_{LL\_neg} - Min_{LL\_neg}\right)}{\left(Max_{neg\_loc} - Min_{neg\_loc}\right)}\right] + Min_{LL\_neg}$$

$$M_{LL\_neg} = \mathbf{6.15} \cdot kip \cdot \frac{ft}{ft}$$

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LRFD TRADITIONAL DECK DESIGN (EQUIVALENT STRIP METHOD) **BEAM SPACING = 12 FEET**

Summary of Moments:

Max Moments between the beams:

Max. Positive Service DC Moment:	$M_{DC\_Pos} := \mathbf{10.502} \cdot kip \cdot \frac{in}{ft}$	$M_{DC\_Pos} = \mathbf{0.875} \cdot kip \cdot \frac{ft}{ft}$	(See STAAD Output)
Max. Negitive Service DC Moment:	$M_{DC\_Neg} := \mathbf{15.418} \cdot kip \cdot \frac{in}{ft}$	$M_{DC\_Neg} = \mathbf{1.285} \cdot kip \cdot \frac{ft}{ft}$	(See STAAD Output)
Max. Positive Service DW Moment:	$M_{DW\_Pos} := \mathbf{1.368} \cdot kip \cdot \frac{in}{ft}$	$M_{DW\_Pos} = \mathbf{0.114} \cdot kip \cdot \frac{ft}{ft}$	(See STAAD Output)
Max. Negitive Service DW Moment:	$M_{DW\_Neg} := \mathbf{2.303} \cdot kip \cdot \frac{in}{ft}$	$M_{DW\_Neg} = \mathbf{0.192} \cdot kip \cdot \frac{ft}{ft}$	(See STAAD Output)

Load Combinations:

Maximum Service I Moment: $M_{Service} := \max\left(M_{DC\_Pos} + M_{DW\_Pos} + M_{LL\_pos}, M_{DC\_Neg} + M_{DW\_Neg} + M_{LL\_neg}\right) \cdot ft$

$M_{Service} = \mathbf{8.999} \cdot kip \cdot ft$

Maximum Strength I Moment: $M_{Strength} := \max\begin{pmatrix} 1.25 M_{DC\_Pos} + \mathbf{1.50} \cdot M_{DW\_Pos} \ldots, 1.25 M_{DC\_Neg} + \mathbf{1.5} \cdot M_{DW\_Neg} \ldots \\ + 1.75 M_{LL\_pos} \qquad\qquad + 1.75 M_{LL\_neg} \end{pmatrix} \cdot ft$

$M_{Strength} = \mathbf{15.282} \cdot kip \cdot ft$

Applied Moment: $M_{applied} := M_{Strength}$ $M_{applied} = \mathbf{15.282} \cdot kip \cdot ft$

Flexure Reinforcement

Minimum Reinforcement: *(AASHTO 5.7.3.3.2 & See AASHTO 5.7.2 for Design Assumptions)*

Modulus of Rapture: *(AASHTO 5.4.2.6 & SDG 1.4.1.B)* $f_r := \mathbf{0.24} \cdot \sqrt{\frac{f_c}{ksi}} \cdot ksi$ $f_r = \mathbf{562.85} \cdot psi$

Moment of Intertia of Slab Section: $I_g := \frac{b \cdot h^{\mathbf{3}}}{\mathbf{12}}$ $I_g = \mathbf{512} \cdot in^{\mathbf{4}}$

Distance from the Extreme Tensile Fiber to the Neutral Axis of the Composite Section: $y_t := \frac{h}{\mathbf{2}}$ $y_t = \mathbf{4} \cdot in$

Cracking Moment: $M_{cr} := \frac{f_r \cdot I_g}{y_t}$ $M_{cr} = \mathbf{6.004} \cdot kip \cdot ft$

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LRFD TRADITIONAL DECK DESIGN (EQUIVALENT STRIP METHOD) **BEAM SPACING = 12 FEET**

Cracking Moment Limit:	$1.2 \cdot M_{cr} = 7.204 \cdot kip \cdot ft$	
Desgin Ultimate Moment:	$M_u := \begin{cases} M_{Strength} & \text{if } M_{Strength} \ge 1.2M_{cr} \\ \min(1.33M_{Strength}, 1.2M_{cr}) & \text{otherwise} \end{cases}$	$M_u = 15.282 \cdot kip \cdot ft$
Distance from Extreme Compressive Fiber to Centroid of Reinforcing Steel:	$d_e := h - cover_b - \frac{d_{bar}(bar_{main})}{2}$	$d_e = 5.688 \cdot in$
Nominal Strength Coefficient of Resistance:	$R_u := \frac{M_u}{\phi_M \cdot b \cdot d_e^2}$	$R_u = 524.938 \cdot psi$
	$m := \frac{f_y}{0.85 \cdot f_c}$	$m = 12.834$
ACI ρ Equation:	$\rho := \frac{1}{m} \cdot \left(1 - \sqrt{1 - \frac{2 \cdot \frac{R_u}{psi} \cdot m}{\frac{f_y}{psi}}}\right)$	$\rho = 0.009305$
	$A_{sreqdpos} := \rho \cdot b \cdot d_e$	$A_{sreqdpos} = 0.635 \cdot in^2$
Minimum Required A_s between beams	$A_{sMinReq} := A_{sreqdpos}$	$A_{sMinReq} = 0.635 \cdot in^2$
Use Main Reinforcing: $bar_{main} \equiv 5$	$Spa_{main} := 5.5in$	
Diameter of Bar:	$d_b := d_{bar}(bar_{main})$	$d_b = 0.625 \cdot in$
Area of Reinforcing in a Section 1 ft Wide:	$A\_s(z, cc) := A_{bar}(z) \cdot \frac{12 \cdot in}{cc}$	
Area of Reinforcing:	$A_{sMain} := A\_s(bar_{main}, Spa_{main})$	$A_{sMain} = 0.676 \cdot in^2$
Depth of Equivalent Rectangular Stress Block:	$a := \frac{A_{sMain} \cdot f_y}{0.85 \cdot f_c \cdot b}$	$a = 0.723 \cdot in$
Ratio of Reinforcement Provided:	$\rho := \frac{A_{sMain}}{b \cdot d_e}$	$\rho = 0.0099$

LRFD TRADITIONAL DECK DESIGN (EQUIVALENT STRIP METHOD) **BEAM SPACING = 12 FEET**

Determine ϕ (Tension or Compression Controlled Section): *(AASHTO 5.7.2.1 & AASHTO 5.7.2.2)*

Determine location of N.A using Whitney Stress Block.:

$$\beta_1 := \begin{cases} \max\left[\left[0.85 - \mathbf{0.05}\cdot\left(\frac{f_c - \mathbf{4}\cdot ksi}{ksi}\right)\right], \mathbf{0.65}\right] & \text{if } f_c > \mathbf{4}ksi \\ \mathbf{0.85} & \text{otherwise} \end{cases} \qquad \beta_1 = \mathbf{0.775}$$

Distance from the Extreme Compression Fiber to the N.A.:

$$c_{comp} := \frac{a}{\beta_1} \qquad c_{comp} = \mathbf{0.933}\cdot in$$

Actual Tensile Strain in Extreme Tension Steel:

$$\varepsilon_T := \mathbf{0.003}\cdot\frac{d_e - c_{comp}}{c_{comp}} \qquad \varepsilon_T = \mathbf{0.015}$$

(AASHTO Figure C5.7.2.1-1)

Comp. and Tension Controlled Section Limits of Net Tensile Strain in the Extreme Tension Steel:

$$\varepsilon_{T\_Limits} := \begin{pmatrix} 0.002 \\ 0.005 \end{pmatrix}$$

Compresssion Controlled if ε_T <or= 0.0002

Tension Controlled if ε_T >or= 0.0005

Comp. and Tension Controlled Reinforced Concrete Section Resistance Factors:

$$\phi := \begin{pmatrix} 0.75 \\ 0.9 \end{pmatrix}$$

Compresssion Controlled

Tension Controlled

(AASHTO 5.5.4.2.1)

Determine Controlling Force:

$$\text{Controlling} := \begin{cases} \textbf{"Compression"} & \text{if } \varepsilon_T \le \varepsilon_{T\_Limits_0} \\ \textbf{"Tension"} & \text{if } \varepsilon_T \ge \varepsilon_{T\_Limits_1} \\ \textbf{"In Transition"} & \text{otherwise} \end{cases} \qquad \text{Controlling} = \textbf{"Tension"}$$

Determine Controlling Resistance Factor:

$$\phi_M := \begin{cases} \phi_0 & \text{if Controlling} = \textbf{"Compression"} \\ \phi_1 & \text{if Controlling} = \textbf{"Tension"} \\ \text{linterp}(\varepsilon_{T\_Limits}, \phi, \varepsilon_T) & \text{otherwise} \end{cases} \qquad \phi_M = \mathbf{0.9}$$

Factored Flexural Resistance:

$$\phi M_n := \phi_M \cdot A_{sMain} \cdot f_y \cdot \left(d_e - \frac{a}{2}\right) \qquad \phi M_n = \mathbf{16.21}\cdot kip\cdot ft$$

Ultimate Moment:

$$M_u = \mathbf{15.282}\cdot kip\cdot ft$$

Check Moment Capacity:

$$\text{CheckMoment} := \begin{cases} \textbf{"OK"} & \text{if } \phi M_n \ge M_u \\ \textbf{"No Good"} & \text{if } \phi M_n < M_u \end{cases} \qquad \text{CheckMoment} = \textbf{"OK"}$$

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LRFD TRADITIONAL DECK DESIGN (EQUIVALENT STRIP METHOD) **BEAM SPACING = 12 FEET**

Crack Control Check

(AASHTO 5.7.3.4)

Thickness of Concrete Cover Measured from Extreme Tension Fiber to Center of the Flexural Reinforcement Located Closest Thereto: $d_c := cover_b + \frac{d_b}{2}$ $d_c = 2.313 \cdot in$

Depth of Neutral Axis $f(x) := \frac{b \cdot x^2}{2 \cdot A_{sMain} \cdot n} + x - d_e$

$x := root(f(x), x, 0, d_e)$ $x = 1.858 \cdot in$

Tensile Stress in Reinforcement at the Service Limit State: $f_{ss} := \frac{M_{Service}}{A_{sMain} \cdot \left(d_e - \frac{x}{3}\right)}$ $f_{ss} = 31.504 \cdot ksi$

Ratio of Flexural Strain at the Extreme Tension Face to the Strain at the Centroid of the Reinforcement Layer Nearest the Tension Face: $\beta_s := 1 + \frac{d_c}{0.7 \cdot (h - d_c)}$ *(AASHTO 5.7.3.4-1)* $\beta_s = 1.581$

Maximum Reinforcement Spacing for Crack Control: $s_{max} := \left(\frac{700 \cdot \gamma_e}{\beta_s \cdot \frac{f_{ss}}{ksi}} - 2 \cdot \frac{d_c}{in}\right) \cdot in$ *(AASHTO 5.7.3.4-1)* $s_{max} = 9.431 \cdot in$

Reiforcement Spacing Provided: $s_{actual} := Spa_{main}$ $s_{actual} = 5.5 \cdot in$

Check Spacing: $CheckSpacing := \begin{cases} \text{"No Good"} & \text{if } s_{actual} > s_{max} \\ \text{"OK"} & \text{otherwise} \end{cases}$ $\boxed{CheckSpacing = \text{"OK"}}$

Distribution Reinforcement

For primary reinforcement perpendicular to traffic *(AASHTO 9.7.3.2)*

Distance Between Beam Flange Tips: $D1 := S_{beam} - b_{tf}$ $D1 = 8\, ft$

Flange Overhang: $D2 := b_{tf} - t_w$ $D2 = 3.417\, ft$

Effective Span Length: $sp_{eff} := D1 + D2$ *(AASHTO 9.7.2.3)* $sp_{eff} = 11.417\, ft$

Distribution Reinforcement %: $DR := \min\left(\frac{220}{\sqrt{\frac{sp_{eff}}{ft}}}, 67\right) \cdot \%$ $DR = 65.111 \cdot \%$

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LRFD TRADITIONAL DECK DESIGN (EQUIVALENT STRIP METHOD) **BEAM SPACING = 12 FEET**

Area of Steel Required: $A_{sDR} := DR \cdot A_{sMain}$ $A_{sDR} = 0.44 \cdot in^2$

Use Dist. Reinforcement: $bar_{DP} := 5$ $Spa_{DP} := 8in$ (Max.)

Area of Reinforcing Provided: $A_{sDP} := A\_s(bar_{DP}, Spa_{DP})$ $A_{sDP} = 0.465 \cdot in^2$

$$CheckDistRein := \begin{cases} \text{"OK"} & \text{if } A_{sDP} \ge A_{sDR} \\ \text{"No Good"} & \text{otherwise} \end{cases}$$

$CheckDistRein = \text{"OK"}$

Temperature & Shrinkage Reinforcement:

(AASHTO 5.10.8.2)

Area of Steel Required for Temp & Shrinkage:

$$A_{s\_ts} := \begin{cases} 0.11\dfrac{in^2}{ft} & \text{if } \dfrac{1.30 \cdot \dfrac{kip}{in \cdot ft} \cdot b \cdot h}{2 \cdot (b+h) \cdot f_y} \le 0.11\dfrac{in^2}{ft} \\ 0.60\dfrac{in^2}{ft} & \text{if } \dfrac{1.30 \cdot \dfrac{kip}{in \cdot ft} \cdot b \cdot h}{2 \cdot (b+h) \cdot f_y} \ge 0.60\dfrac{in^2}{ft} \\ \dfrac{1.30 \cdot \dfrac{kip}{in \cdot ft} \cdot b \cdot h}{2 \cdot (b+h) \cdot f_y} & \text{otherwise} \end{cases}$$

$A_{s\_ts} = 0.11 \cdot \dfrac{in^2}{ft}$

Use: $bar_{TS} := 5$ *(SDG 4.2.11 - #4 Min.)* $Spa_{TS} := 8in$ *(SDG 4.2.11 - 12in Max.)*

Area of Temp and Shrink Reinforcing: $A_{sTS} := A\_s(bar_{TS}, Spa_{TS})$ $A_{sTS} = 0.465 \cdot in^2$

$$Check_{TS} := \begin{cases} \text{"OK"} & \text{if } A_{sDR} \ge A_{s\_ts} \cdot ft \wedge A_{sMain} \ge A_{s\_ts} \cdot ft \wedge A_{sTS} \ge A_{s\_ts} \cdot ft \\ \text{"No Good"} & \text{otherwise} \end{cases}$$

$Check_{TS} = \text{"OK"}$

Maximum Spacing of Temperature and Shrinkage Reinforcement: $s_{TSmax} := \min(3 \cdot h, 18in, 12in)$ $s_{TSmax} = 12 \cdot in$

$$Check_{Spa} := \begin{cases} \text{"OK"} & \text{if } Spa_{DP} \le s_{TSmax} \wedge Spa_{main} \le s_{TSmax} \wedge Spa_{TS} \le s_{TSmax} \\ \text{"No Good"} & \text{otherwise} \end{cases}$$

$Check_{Spa} = \text{"OK"}$

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LRFD TRADITIONAL DECK DESIGN (EQUIVALENT STRIP METHOD) **BEAM SPACING = 12 FEET**

Deck Reinforcement Summary

Main Reinforcement, Transverse (Bot.):	**Use #** $bar_{main} = \mathbf{5}$	**bars at**	$Spa_{main} = \mathbf{5.5} \cdot in$	
Main Reinforcement, Transverse (Top):	**Use #** $bar_{main} = \mathbf{5}$	**bars at**	$Spa_{main} = \mathbf{5.5} \cdot in$	
Distribution Reinforcement, Longitudinal (Bottom):	**Use #** $bar_{DP} = \mathbf{5}$	**bars at**	$Spa_{DP} = \mathbf{8} \cdot in$	(Max.)
Temperature and Shrinkage, Longitudinal (Top):	**Use #** $bar_{TS} = \mathbf{5}$	**bars at**	$Spa_{TS} = \mathbf{8} \cdot in$	(Max.)

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	Part Deck DC & DW Loads		
Job Title Georges El-Gharib Thesis	Ref		
	By GIE	Date 15-Sept-13	Chd
Client UNF	File 12 foot DC Moments.std	Date/Time 16-Sep-2013 20:53	

Job Information

	Engineer	Checked	Approved
Name:	GIE		
Date:	15-Sept-13		

Comments

Finding Dead Load Moments for deck design

Structure Type	PLANE FRAME

Number of Nodes	6	Highest Node	6
Number of Elements	5	Highest Beam	5

Number of Basic Load Cases	2
Number of Combination Load Cases	0

Included in this printout are data for:

All	The Whole Structure

Included in this printout are results for load cases:

Type	L/C	Name
Primary	1	DC COMPONENTS DEAD LOADS
Primary	2	DW FWS DEAD LOAD

Nodes

Node	X (ft)	Y (ft)	Z (ft)
1	0.000	0.000	0.000
2	4.000	0.000	0.000
3	16.000	0.000	0.000
4	28.000	0.000	0.000
5	40.000	0.000	0.000
6	44.000	0.000	0.000

Beams

Beam	Node A	Node B	Length (ft)	Property	β (degrees)
1	2	1	4.000	1	0
2	2	3	12.000	1	0
3	3	4	12.000	1	0
4	4	5	12.000	1	0
5	5	6	4.000	1	0

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	Part Deck DC & DW Loads		
Job Title Georges El-Gharib Thesis	Ref		
	By GIE	Date 15-Sept-13	Chd
Client UNF	File 12 foot DC Moments.std	Date/Time 16-Sep-2013 20:53	

Section Properties

Prop	Section	Area (in^2)	I_{yy} (in^4)	I_{zz} (in^4)	J (in^4)	Material
1	Rect 8.00x12.00	96.000	1.15E+3	512.008	1.2E+3	CONCRETE

Supports

Node	X (kip/in)	Y (kip/in)	Z (kip/in)	rX (kip ft/deg)	rY (kip ft/deg)	rZ (kip ft/deg)
2	Fixed	Fixed	Fixed	-	-	-
3	-	Fixed	-	-	-	-
4	-	Fixed	-	-	-	-
5	Fixed	Fixed	Fixed	-	-	-

Basic Load Cases

Number	Name
1	DC COMPONENTS DEAD LOADS
2	DW FWS DEAD LOAD

Beam Maximum Moments

Distances to maxima are given from beam end A.

Beam	Node A	Length (ft)	L/C		d (ft)	Max My (kip in)	d (ft)	Max Mz (kip in)
1	2	4.000	1:DC COMPOI	Max -ve	0.000	0.000	0.000	26.640
				Max +ve	0.000	0.000	4.000	-0.000
			2:DW FWS DE	Max -ve	0.000	0.000	0.000	1.440
				Max +ve	0.000	0.000		
2	2	12.000	1:DC COMPOI	Max -ve	0.000	0.000	0.000	26.640
				Max +ve	0.000	0.000	7.000	-5.106
			2:DW FWS DE	Max -ve	0.000	0.000	12.000	2.303
				Max +ve	0.000	0.000	6.000	-1.368
3	3	12.000	1:DC COMPOI	Max -ve	0.000	0.000	0.000	15.418
				Max +ve	0.000	0.000	6.000	-10.502
			2:DW FWS DE	Max -ve	0.000	0.000	0.000	2.303
				Max +ve	0.000	0.000	6.000	-0.937
4	4	12.000	1:DC COMPOI	Max -ve	0.000	0.000	12.000	26.640
				Max +ve	0.000	0.000	5.000	-5.106
			2:DW FWS DE	Max -ve	0.000	0.000	0.000	2.303
				Max +ve	0.000	0.000	6.000	-1.368
5	5	4.000	1:DC COMPOI	Max -ve	0.000	0.000	0.000	26.640
				Max +ve	0.000	0.000		
			2:DW FWS DE	Max -ve	0.000	0.000	0.000	1.440
				Max +ve	0.000	0.000	4.000	-0.000

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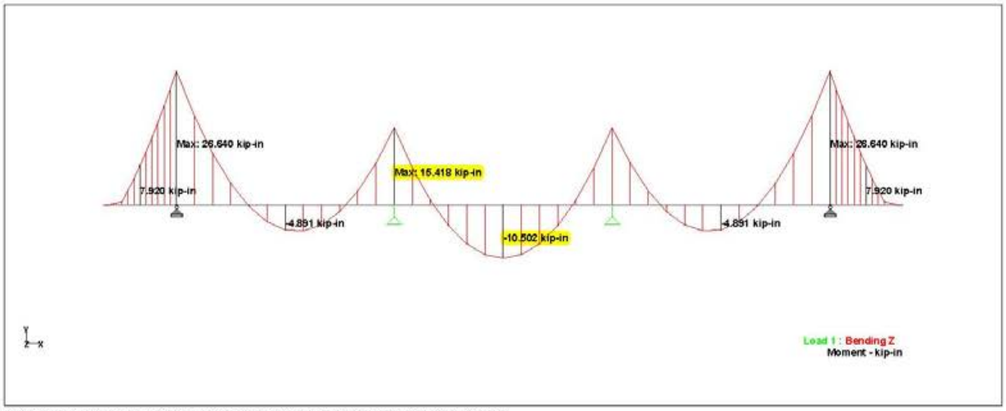
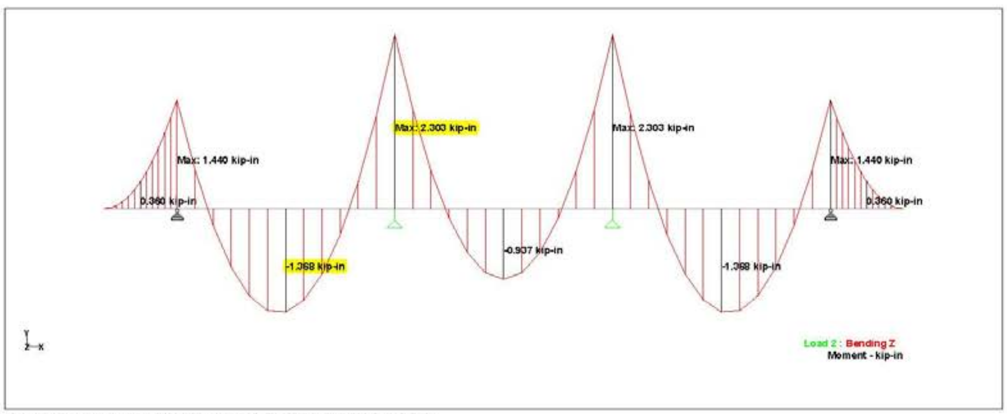
	Part Deck DC & DW Loads		
Job Title Georges El-Gharib Thesis	Ref		
	By GIE	Date 15-Sept-13	Chd
Client UNF	File 12 foot DC Moments.std	Date/Time 16-Sep-2013 20:53	



Whole Structure Mz 3.6kip-in:1ft 1 DC COMPONENTS DEAD LOADS



Whole Structure Mz 0.24kip-in:1ft 2 DW FWS DEAD LOAD

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Appendix E: Finite element model sample, 6-foot beam spacing

FINITE ELEMENT DECK DESIGN **BEAM SPACING = 6 FEET**

Codes and Specification

- "AASHTO LRFD Bridge Design Specifications "American Association of State Highway and Transportation Officials (AASHTO), 6th Edition, 2012 with Interims through 2013.
- "Florida Department of Transportation Structures Design Guidelines for Load and Resistance Factor Design," January 2013 Edition.
- "Florida Department of Transportation Design Standards", 2014.

Reinforcing Dimensions

Input

(NOTE: For deck slabs, use same reinforcement Top & Bot. Do not include Integral Wearing Surface in "h" OR "cover")

Variables:

Beam Type:	Beam := **"FIB36"**	(Use 2, 3, 4, 5, 6, FBT72, FBT78, FIB36, FIB45, FIB54, FIB63, FIB72, FIB78)
Concrete Strength:	$f_c := \mathbf{5500psi}$	*(Assume Extremely Aggr Env., use Class IV CIP Bridge Deck Concrete, SDG Table 1.4.3-1)*
Concrete Weight:	$w_c := 150 \cdot pcf$	*(SDG Table 2.2-1)*
Aggregate Correction Factor:	$K_1 := 0.90$	*(SDG 1.4.1.A)*
Yield Strength of Reinforcing Steel:	$f_y := 60ksi$	
Width of Design Section:	$b := 1ft$	*(SDG 4.2.2.B - For new construction of "Short Bridges" other than Inverted-T Beam bridge superstructures, the minimum thickness of bridge decks cast-in-place (CIP) on beams or girders is 8-inches.*
Height of Design Section (deck thickness):	$h := \mathbf{7.0in}$	
Height of Sacrificial Wearing Surface:	$h_{sac} := 0.0in$	
Load Reduction Factor for Moment (Initial Guess):	$\phi_M := 0.90$	
Top cover:	$cover_t := \mathbf{1.5in}$	*(SDG Table 1.4.2-1)*
Bottom cover:	$cover_b := \mathbf{1.5in}$	*(SDG Table 1.4.2-1)*
Exposure Condition:	exposure_condition := **"Class 1"**	*(SDG 4.1.8)*
Beam Spacing:	$S_{beam} := \mathbf{6}ft + \mathbf{0.0in}$	
Future Wearing Surface:	$FWS := \mathbf{15psf}$	*(SDG Table 2.2-1)*
Weight of SIP Forms:	$SIP := 20psf$	
Width of Traffic Railing Barrier:	$W_{barr} := 1.5ft$	
Width of Raised Sidewalk:	$W_{sw} := 0ft$	
Traffic Railing Barrier:	$w_{barrier} := 420plf$	
Weight of Median:	$w_{med} := \mathbf{0}ft \cdot b \cdot w_c$	$w_{med} = \mathbf{0} \cdot plf$
Bridge Skew:	$Skew := mean[(\mathbf{0}),(\mathbf{0})]\, deg$	$Skew = \mathbf{0} \cdot deg$

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FINITE ELEMENT DECK DESIGN **BEAM SPACING = 6 FEET**

Weight of Sidewalk:	$w_{sw} := \boxed{0in + (W_{barr} + W_{sw})\cdot 0.00}\cdot(W_{barr} + W_{sw})\cdot w_c$	$w_{sw} = \mathbf{0}\cdot plf$	
Weight of Pedestrian/Bicycle Railing and Fence:	$w_{barr\_ped} := 0plf + 0plf$	$w_{barr\_ped} = \mathbf{0}\cdot plf$	

Beam Properties

Constants:

Top Flange Width:	$b_{tf} = \mathbf{48}\cdot in$		
Web Width:	$t_w = \mathbf{7}\cdot in$		
Concrete Unit Weight for Modulus of Elasticity Calc:	$\gamma_c := \mathbf{145}pcf$		*(SDG 1.4.1.A)*
Modulus of Elasticity Deck:	$E_c := \mathbf{33000}\cdot K_1\cdot\left(\frac{\gamma_c}{kcf}\right)^{\mathbf{1.5}}\cdot\sqrt{\frac{f_c}{ksi}}\cdot ksi$	$E_c = \mathbf{3845.8}\cdot ksi$	*(AASHTO 5.4.2.4)*
Modulus of Elasticity Reinforcing Steel:	$E_s := \mathbf{29000}ksi$		*(AASHTO 5.4.3.2)*
Modular Ratio:	$n := \mathrm{round}\left(\frac{E_s}{E_c}\right)$	$n = \mathbf{8}$	*(AASHTO 5.7.1)*
Area of Deck Section:	$A_c := h\cdot b$	$A_c = \mathbf{84}\cdot in^2$	
Crack Control Exposure Condition Factor:	$\gamma_e := \begin{cases} \mathbf{1.00} & \text{if exposure\_condition} = \text{"Class 1"} \\ \mathbf{0.75} & \text{otherwise} \end{cases}$	$\gamma_e = \mathbf{1}$	*(AASHTO 5.7.3.4)*

Design Moment

Location of Negative Live Load Design Moment:

AASHTO LRFD 4.6.2.1.6 --> The design section for negative moments and shear forces, where investigated, may be taken as follows:

For precast I-shaped concrete beams ....crossection (k) from Table 4.6.2.2.1-1: one-third the flange width, but not exceeding 15.0 in from the centerline of support.

The negative live load design moment is taken at a distance from the supports: $Loc_{negative} := \min\left(\frac{1}{3}\cdot b_{tf}, \mathbf{15}\cdot in\right)$ $Loc_{negative} = \mathbf{15.0}\cdot in$

AASHTO Table A4-1 - Deck Slab Design Table

Dead Load Moments for Moment Analysis:

"DC" loads include the dead load of structural components and non-structural attachments

Self-weight of Deck Slab:	$w_{slab} := \left[(h + h_{sac})\cdot b\right]\cdot w_c$	$w_{slab} = \mathbf{0.088}\cdot klf$
Weight of Traffic Railing Barriers:	$P_{barrier} := w_{barrier}\cdot b$	$P_{barrier} = \mathbf{0.420}\cdot kip$

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FINITE ELEMENT DECK DESIGN **BEAM SPACING = 6 FEET**

Weight of Pedestrian/Bicycle Railing and Fence:	$P_{barTypeK} := w_{barr\_ped} \cdot b$	$P_{barTypeK} = \mathbf{0.000} \cdot kip$
Weight of Median:	$w_{median} := w_{med}$	$w_{median} = \mathbf{0.000} \cdot klf$
Weight of Sidewalk:	$w_{sw} := \dfrac{w_{sw} \cdot b}{W_{sw} + W_{barr}}$	$w_{sw} = \mathbf{0.000} \cdot klf$
Stay-in-Place Forms:	$w_{sip} := SIP \cdot b$	$w_{sip} = \mathbf{0.020} \cdot klf$

"DW" loads include the dead load of a future wearing surface and utilities

Weight of Future Wearing Surface:	$w_{fws} := FWS \cdot b$	$w_{fws} = \mathbf{0.015} \cdot klf$

Max. Positive Live Load Moment:

From Finite Element Model: $M_{LL\_pos} := \mathbf{2.56} \cdot kip \cdot \dfrac{ft}{ft}$ $M_{LL\_pos} = \mathbf{2.56} \cdot kip \cdot \dfrac{ft}{ft}$

Max. Negative Live Load Moment:

From Finite Element Model:

From Finite Element Models $M_{LL\_neg} := \mathbf{1.14} \cdot kip \cdot \dfrac{ft}{ft}$

$M_{LL\_neg} = \mathbf{1.14} \cdot kip \cdot \dfrac{ft}{ft}$

FINITE ELEMENT DECK DESIGN **BEAM SPACING = 6 FEET**

Summary of Moments:

Max Moments between the beams:

Max. Positive Service DC Moment: $M_{DC\_Pos} := \mathbf{0.315} \cdot kip \cdot \frac{ft}{ft}$ $M_{DC\_Pos} = \mathbf{0.315} \cdot kip \cdot \frac{ft}{ft}$ (See STAAD Output)

Max. Negative Service DC Moment: $M_{DC\_Neg} := \mathbf{0.103} \cdot kip \cdot \frac{ft}{ft}$ $M_{DC\_Neg} = \mathbf{0.103} \cdot kip \cdot \frac{ft}{ft}$ (See STAAD Output)

Max. Positive Service DW Moment: $M_{DW\_Pos} := \mathbf{0.064} \cdot kip \cdot \frac{ft}{ft}$ $M_{DW\_Pos} = \mathbf{0.064} \cdot kip \cdot \frac{ft}{ft}$ (See STAAD Output)

Max. Negative Service DW Moment $M_{DW\_Neg} := \mathbf{0.000} \cdot kip \cdot \frac{ft}{ft}$ $M_{DW\_Neg} = \mathbf{0} \cdot kip \cdot \frac{ft}{ft}$ (See STAAD Output)

Load Combinations:

Maximum Service I Moment: $M_{Service} := \max(M_{DC\_Pos} + M_{DW\_Pos} + M_{LL\_pos}, M_{DC\_Neg} + M_{DW\_Neg} + M_{LL\_neg}) \cdot ft$

$M_{Service} = \mathbf{2.939} \cdot kip \cdot ft$

Maximum Strength I Moment: $M_{Strength} := \max\begin{pmatrix} 1.25M_{DC\_Pos} + \mathbf{1.50} \cdot M_{DW\_Pos} \cdots, 1.25M_{DC\_Neg} + \mathbf{1.5} \cdot M_{DW\_Neg} \cdots \\ +1.75M_{LL\_pos} \qquad\qquad +1.75M_{LL\_neg} \end{pmatrix} \cdot ft$

$M_{Strength} = \mathbf{4.97} \cdot kip \cdot ft$

Applied Moment: $M_{applied} := M_{Strength}$ $M_{applied} = \mathbf{4.97} \cdot kip \cdot ft$

Flexure Reinforcement

Minimum Reinforcement: *(AASHTO 5.7.3.3.2 & See AASHTO 5.7.2 for Design Assumptions)*

Modulus of Rapture: *(AASHTO 5.4.2.6 & SDG 1.4.1.B)* $f_r := \mathbf{0.24} \cdot \sqrt{\frac{f_c}{ksi}} \cdot ksi$ $f_r = \mathbf{562.85} \cdot psi$

Moment of Inertia of Slab Section: $I_g := \frac{b \cdot h^{\mathbf{3}}}{\mathbf{12}}$ $I_g = \mathbf{343} \cdot in^{\mathbf{4}}$

Distance from the Extreme Tensile Fiber to the Neutral Axis of the Composite Section: $y_t := \frac{h}{\mathbf{2}}$ $y_t = \mathbf{3.5} \cdot in$

Cracking Moment: $M_{cr} := \frac{f_r \cdot I_g}{y_t}$ $M_{cr} = \mathbf{4.597} \cdot kip \cdot ft$

FINITE ELEMENT DECK DESIGN **BEAM SPACING = 6 FEET**

Cracking Moment Limit:	$1.2 \cdot M_{cr} = 5.516 \cdot kip \cdot ft$	
Des gin Ultimate Moment:	$M_u := \begin{cases} M_{Strength} & \text{if } M_{Strength} \geq 1.2M_{cr} \\ \min(1.33M_{Strength}, 1.2M_{cr}) & \text{otherwise} \end{cases}$	$M_u = 5.516 \cdot kip \cdot ft$
Distance from Extreme Compressive Fiber to Centroid of Reinforcing Steel:	$d_e := h - cover_b - \frac{d_{bar}(bar_{main})}{2}$	$d_e = 5.188 \cdot in$
Nominal Strength Coefficient of Resistance:	$R_u := \frac{M_u}{\phi_M \cdot b \cdot d_e^2}$	$R_u = 227.751 \cdot psi$
	$m := \frac{f_y}{0.85 \cdot f_c}$	$m = 12.834$
ACI ρ Equation:	$\rho := \frac{1}{m} \cdot \left(1 - \sqrt{1 - \frac{2 \cdot \frac{R_u}{psi} \cdot m}{\frac{f_y}{psi}}}\right)$	$\rho = 0.003893$
	$A_{sreqdpos} := \rho \cdot b \cdot d_e$	$A_{sreqdpos} = 0.242 \cdot in^2$
Minimum Required A_s between beams	$A_{sMinReq} := A_{sreqdpos}$	$A_{sMinReq} = 0.242 \cdot in^2$
Use Main Reinforcing: $bar_{main} \equiv 5$	$Spa_{main} := 12.00in$	
Diameter of Bar:	$d_b := d_{bar}(bar_{main})$	$d_b = 0.625 \cdot in$
Area of Reinforcing in a Section 1 ft Wide:	$A\_s(z, cc) := A_{bar}(z) \cdot \frac{12 \cdot in}{cc}$	
Area of Reinforcing:	$A_{sMain} := A\_s(bar_{main}, Spa_{main})$	$A_{sMain} = 0.31 \cdot in^2$
Depth of Equivalent Rectangular Stress Block:	$a := \frac{A_{sMain} \cdot f_y}{0.85 \cdot f_c \cdot b}$	$a = 0.332 \cdot in$
Ratio of Reinforcement Provided:	$\rho := \frac{A_{sMain}}{b \cdot d_e}$	$\rho = 0.005$

FINITE ELEMENT DECK DESIGN **BEAM SPACING = 6 FEET**

Determine ϕ (Tension or Compression Controlled Section): *(AASHTO 5.7.2.1 & AASHTO 5.7.2.2)*

Determine location of N.A using Whitney Stress Block.:

$$\beta_1 := \begin{cases} \max\left[\left[0.85 - \mathbf{0.05}\cdot\left(\dfrac{f_c - \mathbf{4}\cdot ksi}{ksi}\right)\right], \mathbf{0.65}\right] & \text{if } f_c > \mathbf{4}ksi \\ \mathbf{0.85} & \text{otherwise} \end{cases}$$

$\beta_1 = \mathbf{0.775}$

Distance from the Extreme Compression Fiber to the N.A.:

$$c_{comp} := \frac{a}{\beta_1}$$

$c_{comp} = \mathbf{0.428}\cdot in$

Actual Tensile Strain in Extreme Tension Steel:

$$\varepsilon_T := \mathbf{0.003}\cdot\frac{d_e - c_{comp}}{c_{comp}}$$

(AASHTO Figure C5.7.2.1-1)

$\varepsilon_T = \mathbf{0.033}$

Comp. and Tension Controlled Section Limits of Net Tensile Strain in the Extreme Tension Steel:

$$\varepsilon_{T\_Limits} := \begin{pmatrix} 0.002 \\ 0.005 \end{pmatrix}$$

Compression Controlled if ε_T <or= 0.0002

Tension Controlled if ε_T >or= 0.0005

Comp. and Tension Controlled Reinforced Concrete Section Resistance Factors:

$$\phi := \begin{pmatrix} 0.75 \\ \mathbf{0.90} \end{pmatrix}$$

Compression Controlled

Tension Controlled

(AASHTO 5.5.4.2.1)

Determine Controlling Force:

$$\text{Controlling} := \begin{cases} \textbf{"Compression"} & \text{if } \varepsilon_T \le \varepsilon_{T\_Limits_0} \\ \textbf{"Tension"} & \text{if } \varepsilon_T \ge \varepsilon_{T\_Limits_1} \\ \textbf{"In Transition"} & \text{otherwise} \end{cases}$$

Controlling = **"Tension"**

Determine Controlling Resistance Factor:

$$\phi_M := \begin{cases} \phi_{\mathbf{0}} & \text{if Controlling} = \textbf{"Compression"} \\ \phi_{\mathbf{1}} & \text{if Controlling} = \textbf{"Tension"} \\ \text{linterp}(\varepsilon_{T\_Limits}, \phi, \varepsilon_T) & \text{otherwise} \end{cases}$$

$\phi_M = \mathbf{0.9}$

Factored Flexural Resistance:

$$\phi M_n := \phi_M \cdot A_{sMain} \cdot f_y \cdot \left(d_e - \frac{a}{\mathbf{2}}\right)$$

$\phi M_n = \mathbf{7.005}\cdot kip\cdot ft$

Ultimate Moment:

$M_u = \mathbf{5.516}\cdot kip\cdot ft$

Check Moment Capacity:

$$\text{CheckMoment} := \begin{cases} \textbf{"OK"} & \text{if } \phi M_n \ge M_u \\ \textbf{"No Good"} & \text{if } \phi M_n < M_u \end{cases}$$

$\boxed{\text{CheckMoment} = \textbf{"OK"}}$

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FINITE ELEMENT DECK DESIGN **BEAM SPACING = 6 FEET**

Crack Control Check

(AASHTO 5.7.3.4)

Thickness of Concrete Cover Measured from Extreme Tension Fiber to Center of the Flexural Reinforcement Located Closest Thereto: $d_c := cover_b + \frac{d_b}{2}$ $\quad d_c = 1.812 \cdot in$

Depth of Neutral Axis $f(x) := \frac{b \cdot x^2}{2 \cdot A_{sMain} \cdot n} + x - d_e$

$x := root(f(x), x, 0, d_e)$ $\quad x = 1.272 \cdot in$

Tensile Stress in Reinforcement at the Service Limit State: $f_{ss} := \frac{M_{Service}}{A_{sMain} \cdot \left(d_e - \frac{x}{3}\right)}$ $\quad f_{ss} = 23.883 \cdot ksi$

Ratio of Flexural Strain at the Extreme Tension Face to the Strain at the Centroid of the Reinforcement Layer Nearest the Tension Face: $\beta_s := 1 + \frac{d_c}{0.7 \cdot (h - d_c)}$ *(AASHTO 5.7.3.4-1)* $\quad \beta_s = 1.499$

Maximum Reinforcement Spacing for Crack Control: $s_{max} := \left(\frac{700 \cdot \gamma_e}{\beta_s \cdot \frac{f_{ss}}{ksi}} - 2 \cdot \frac{d_c}{in}\right) \cdot in$ *(AASHTO 5.7.3.4-1)* $\quad s_{max} = 15.926 \cdot in$

Reinforcement Spacing Provided: $s_{actual} := Spa_{main}$ $\quad s_{actual} = 12 \cdot in$

Check Spacing: $CheckSpacing := \begin{cases} \textbf{"No Good"} & \text{if } s_{actual} > s_{max} \\ \textbf{"OK"} & \text{otherwise} \end{cases}$ $\quad CheckSpacing = \textbf{"OK"}$

Distribution Reinforcement

For primary reinforcement perpendicular to traffic *(AASHTO 9.7.3.2)*

Distance Between Beam Flange Tips: $D1 := S_{beam} - b_{tf}$ $\quad D1 = 2\,ft$

Flange Overhang: $D2 := b_{tf} - t_w$ $\quad D2 = 3.417\,ft$

Effective Span Length: $sp_{eff} := D1 + D2$ *(AASHTO 9.7.2.3)* $\quad sp_{eff} = 5.417\,ft$

Distribution Reinforcement %: $DR := \min\left(\frac{220}{\sqrt{\frac{sp_{eff}}{ft}}}, 67\right) \cdot \%$ $\quad DR = 67 \cdot \%$

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FINITE ELEMENT DECK DESIGN **BEAM SPACING = 6 FEET**

Area of Steel Required: $A_{sDR} := DR \cdot A_{sMain}$ $A_{sDR} = \mathbf{0.208} \cdot in^2$

Use Dist. Reinforcement: $bar_{DP} := 5$ $Spa_{DP} := \mathbf{12.0} in$ (Max.)

Area of Reinforcing Provided: $A_{sDP} := A\_s(bar_{DP}, Spa_{DP})$ $A_{sDP} = \mathbf{0.31} \cdot in^2$

$$CheckDistRein := \begin{cases} \text{"OK"} & \text{if } A_{sDP} \ge A_{sDR} \\ \text{"No Good"} & \text{otherwise} \end{cases}$$

$CheckDistRein = \text{"OK"}$

Temperature & Shrinkage Reinforcement:

(AASHTO 5.10.8.2)

Area of Steel Required for Temp & Shrinkage:

$$A_{s\_ts} := \begin{cases} 0.11 \dfrac{in^2}{ft} & \text{if } \dfrac{1.30 \cdot \dfrac{kip}{in \cdot ft} \cdot b \cdot h}{2 \cdot (b + h) \cdot f_y} \le 0.11 \dfrac{in^2}{ft} \\ 0.60 \dfrac{in^2}{ft} & \text{if } \dfrac{1.30 \cdot \dfrac{kip}{in \cdot ft} \cdot b \cdot h}{2 \cdot (b + h) \cdot f_y} \ge 0.60 \dfrac{in^2}{ft} \\ \dfrac{1.30 \cdot \dfrac{kip}{in \cdot ft} \cdot b \cdot h}{2 \cdot (b + h) \cdot f_y} & \text{otherwise} \end{cases}$$

$A_{s\_ts} = \mathbf{0.11} \cdot \dfrac{in^2}{ft}$

Use: $bar_{TS} := 5$ *(SDG 4.2.11 - #4 Min.)* $Spa_{TS} := \mathbf{12.0} in$ *(SDG 4.2.11 - 12in Max.)*

Area of Temp and Shrink Reinforcing: $A_{sTS} := A\_s(bar_{TS}, Spa_{TS})$ $A_{sTS} = \mathbf{0.31} \cdot in^2$

$$Check_{TS} := \begin{cases} \text{"OK"} & \text{if } A_{sDR} \ge A_{s\_ts} \cdot ft \wedge A_{sMain} \ge A_{s\_ts} \cdot ft \wedge A_{sTS} \ge A_{s\_ts} \cdot ft \\ \text{"No Good"} & \text{otherwise} \end{cases}$$

$Check_{TS} = \text{"OK"}$

Maximum Spacing of Temperature and Shrinkage Reinforcement: $s_{TSmax} := \min(3 \cdot h, 18 in, 12 in)$ $s_{TSmax} = \mathbf{12} \cdot in$

$$Check_{Spa} := \begin{cases} \text{"OK"} & \text{if } Spa_{DP} \le s_{TSmax} \wedge Spa_{main} \le s_{TSmax} \wedge Spa_{TS} \le s_{TSmax} \\ \text{"No Good"} & \text{otherwise} \end{cases}$$

$Check_{Spa} = \text{"OK"}$

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FINITE ELEMENT DECK DESIGN **BEAM SPACING = 6 FEET**

Deck Reinforcement Summary

Main Reinforcement, Transverse (Bot.):	**Use #**	$bar_{main} = 5$	**bars at**	$Spa_{main} = 12 \cdot in$	
Main Reinforcement, Transverse (Top):	**Use #**	$bar_{main} = 5$	**bars at**	$Spa_{main} = 12 \cdot in$	
Distribution Reinforcement, Longitudinal (Bottom):	**Use #**	$bar_{DP} = 5$	**bars at**	$Spa_{DP} = 12 \cdot in$	(Max.)
Temperature and Shrinkage, Longitudinal (Top):	**Use #**	$bar_{TS} = 5$	**bars at**	$Spa_{TS} = 12 \cdot in$	(Max.)

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Job Information

	Engineer	Checked	Approved
Name:	GIE		
Date:	05-July-13		

Comments

FIB 36

Structure Type	SPACE FRAME

Number of Nodes	842	Highest Node	842
Number of Elements	345	Highest Beam	1115
Number of Plates	770	Highest Plate	824

Number of Basic Load Cases	3
Number of Combination Load Cases	1

Included in this printout are data for:

All	The Whole Structure

Included in this printout are results for load cases:

Type	L/C	Name
Generation	2	LOAD GENERATION, LOAD #2, (1 of 21)
Generation	3	LOAD GENERATION, LOAD #3, (2 of 21)
Generation	4	LOAD GENERATION, LOAD #4, (3 of 21)
Generation	5	LOAD GENERATION, LOAD #5, (4 of 21)
Generation	6	LOAD GENERATION, LOAD #6, (5 of 21)
Generation	7	LOAD GENERATION, LOAD #7, (6 of 21)
Generation	8	LOAD GENERATION, LOAD #8, (7 of 21)
Generation	9	LOAD GENERATION, LOAD #9, (8 of 21)
Generation	10	LOAD GENERATION, LOAD #10, (9 of 21)
Generation	11	LOAD GENERATION, LOAD #11, (10 of 2
Generation	12	LOAD GENERATION, LOAD #12, (11 of 2
Generation	13	LOAD GENERATION, LOAD #13, (12 of 2
Generation	14	LOAD GENERATION, LOAD #14, (13 of 2
Generation	15	LOAD GENERATION, LOAD #15, (14 of 2
Generation	16	LOAD GENERATION, LOAD #16, (15 of 2
Generation	17	LOAD GENERATION, LOAD #17, (16 of 2
Generation	18	LOAD GENERATION, LOAD #18, (17 of 2
Generation	19	LOAD GENERATION, LOAD #19, (18 of 2
Generation	20	LOAD GENERATION, LOAD #20, (19 of 2
Generation	21	LOAD GENERATION, LOAD #21, (20 of 2
Generation	22	LOAD GENERATION, LOAD #22, (21 of 2
Generation	23	LOAD GENERATION, LOAD #23, (1 of 13)
Generation	24	LOAD GENERATION, LOAD #24, (2 of 13)
Generation	25	LOAD GENERATION, LOAD #25, (3 of 13)

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Job Information Cont...

Type	L/C	Name
Generation	26	LOAD GENERATION, LOAD #26, (4 of 13)
Generation	27	LOAD GENERATION, LOAD #27, (5 of 13)
Generation	28	LOAD GENERATION, LOAD #28, (6 of 13)
Generation	29	LOAD GENERATION, LOAD #29, (7 of 13)
Generation	30	LOAD GENERATION, LOAD #30, (8 of 13)
Generation	31	LOAD GENERATION, LOAD #31, (9 of 13)
Generation	32	LOAD GENERATION, LOAD #32, (10 of 1:
Generation	33	LOAD GENERATION, LOAD #33, (11 of 1:
Generation	34	LOAD GENERATION, LOAD #34, (12 of 1:
Generation	35	LOAD GENERATION, LOAD #35, (13 of 1:

Section Properties

Prop	Section	Area (in^2)	I_{yy} (in^4)	I_{zz} (in^4)	J (in^4)	Material
3	Rect 36.00x36.00	1.3E+3	140E+3	140E+3	236E+3	CONCRETE
4	Rect 18.00x18.00	324.000	8.75E+3	8.75E+3	14.8E+3	CONCRETE
5	FIB-36	810.187	81.4E+3	128E+3	31.1E+3	CONCRETE
6	TYPEFBARRIERL	401.375	6.5E+3	36E+3	14.1E+3	CONCRETE
7	TYPEFBARRIER	403.781	6.65E+3	36.1E+3	14.4E+3	CONCRETE

Plate Thickness

Prop	Node A (in)	Node B (in)	Node C (in)	Node D (in)	Material
1	7.000	7.000	7.000	7.000	CONCRETE
2	12.000	12.000	12.000	12.000	CONCRETE

Materials

Mat	Name	E (kip/in^2)	ν	Density (kip/in^3)	α (/°F)
1	STEEL	29E+3	0.300	0.000	6E-6
2	STAINLESSSTEEL	28E+3	0.300	0.000	10E-6
3	ALUMINUM	10E+3	0.330	0.000	13E-6
4	CONCRETE	3.15E+3	0.170	0.000	5E-6

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Supports

Node	X (kip/in)	Y (kip/in)	Z (kip/in)	rX (kip°ft/deg)	rY (kip°ft/deg)	rZ (kip°ft/deg)
1	Fixed	Fixed	Fixed	Fixed	Fixed	Fixed
2	Fixed	Fixed	Fixed	Fixed	Fixed	Fixed
3	Fixed	Fixed	Fixed	Fixed	Fixed	Fixed
4	Fixed	Fixed	Fixed	Fixed	Fixed	Fixed
5	Fixed	Fixed	Fixed	Fixed	Fixed	Fixed
6	Fixed	Fixed	Fixed	Fixed	Fixed	Fixed
7	Fixed	Fixed	Fixed	Fixed	Fixed	Fixed
813	Fixed	Fixed	Fixed	Fixed	Fixed	Fixed
814	Fixed	Fixed	Fixed	Fixed	Fixed	Fixed
815	Fixed	Fixed	Fixed	Fixed	Fixed	Fixed
816	Fixed	Fixed	Fixed	Fixed	Fixed	Fixed
817	Fixed	Fixed	Fixed	Fixed	Fixed	Fixed
818	Fixed	Fixed	Fixed	Fixed	Fixed	Fixed
819	Fixed	Fixed	Fixed	Fixed	Fixed	Fixed

Basic Load Cases

Number	Name
1	SELFWEIGHT
36	STAY IN PLACE FORMS
37	FUTURE WEARING SURFACE

Moving Loads: Loads 2 to 22

	Initial Position			Increment			
Type	X (ft)	Y (ft)	Z (ft)	X (ft)	Y (ft)	Z (ft)	Range (ft)
1	0.000	0.000	21.000	2.000	-	-	-

Moving Loads: Loads 23 to 35

There is no data of this type - Analysis results are not available

Beam Displacement Detail Summary

Displacements shown in italic indicate the presence of an offset

	Beam	L/C	d (ft)	X (in)	Y (in)	Z (in)	Resultant (in)
Max X	829	24:LOAD GENI	2.000	**0.010**	-0.097	0.002	0.098
Min X	978	24:LOAD GENI	1.800	**-0.010**	-0.099	0.002	0.100
Max Y	15	30:LOAD GENI	4.000	*0.003*	**0.002**	-0.003	0.005
Min Y	907	24:LOAD GENI	1.000	-0.000	**-0.220**	0.002	0.220
Max Z	904	24:LOAD GENI	0.000	*0.000*	*-0.003*	**0.005**	0.006

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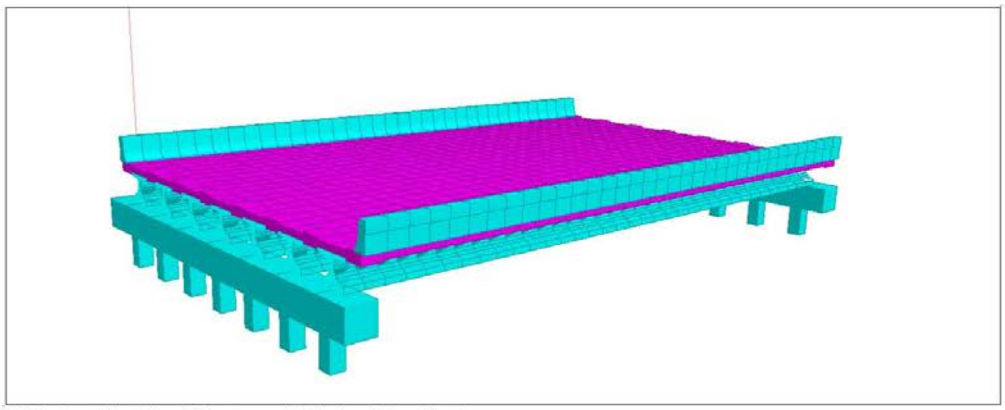
Plate Centre Stress Summary

			Shear		Membrane			Bending		
	Plate	L/C	Qx (psi)	Qy (psi)	Sx (psi)	Sy (psi)	Sxy (psi)	Mx (lb·in/in)	My (lb·in/in)	Mxy (lb·in/in)
Max Qx	309	24:LOAD GENI	**7.186**	8.117	-140.322	20.477	59.473	647.441	2.13E+3	-183.724
Min Qx	535	24:LOAD GENI	**-7.186**	8.117	-140.317	20.477	-59.474	647.426	2.13E+3	183.728
Max Qy	498	33:LOAD GENI	0.419	**11.842**	-177.033	27.961	-24.218	821.577	2.32E+3	123.884
Min Qy	492	24:LOAD GENI	0.419	**-11.842**	-177.083	27.928	24.215	821.583	2.32E+3	-123.921
Max Sx	41	24:LOAD GENI	3.977	-0.104	**157.934**	-14.847	99.738	-298.015	43.113	-240.961
Min Sx	489	24:LOAD GENI	0.460	-1.384	**-190.690**	28.098	3.269	866.888	2.29E+3	85.991
Max Sy	802	23:LOAD GENI	-3.226	1.296	1.802	**87.036**	-31.371	-129.181	-216.881	21.860
Min Sy	485	23:LOAD GENI	5.905	-0.006	11.379	**-125.651**	26.066	611.658	343.777	-146.144
Max Sxy	41	24:LOAD GENI	3.977	-0.104	157.934	-14.847	**99.738**	-298.015	43.113	-240.961
Min Sxy	806	24:LOAD GENI	-3.977	-0.104	157.933	-14.846	**-99.737**	-298.013	43.112	240.958
Max Mx	490	24:LOAD GENI	-0.262	-0.270	-177.299	23.835	-1.438	**915.246**	2.22E+3	-1.144
Min Mx	417	24:LOAD GENI	3.143	-0.000	2.744	-101.776	0.000	**-489.511**	-1.345	-0.002
Max My	421	24:LOAD GENI	0.000	10.154	-165.133	27.900	-0.000	779.696	**2.56E+3**	0.003
Min My	382	24:LOAD GENI	0.391	-10.809	-129.564	15.772	-9.207	39.481	**-1.14E+3**	111.896
Max Mxy	256	33:LOAD GENI	-0.745	-4.773	-65.214	4.515	-47.371	195.137	572.249	**467.438**
Min Mxy	616	33:LOAD GENI	0.745	-4.773	-65.212	4.514	47.370	195.132	572.246	**-467.436**



3D Rendered View (Input data was modified after picture taken)

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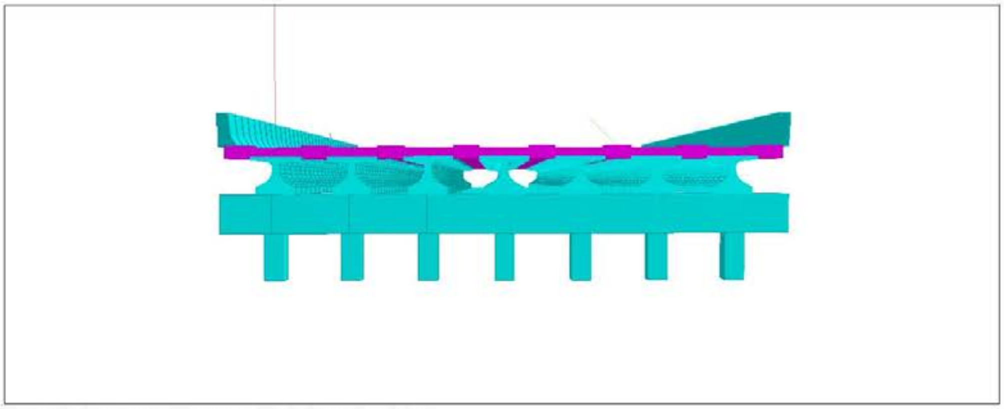
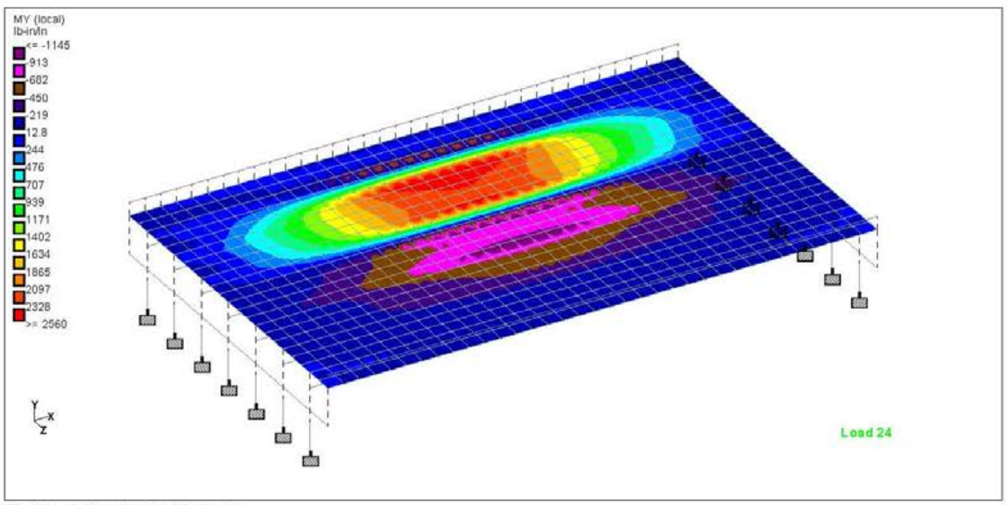


Typical Section (Input data was modified after picture taken)



Positive & Negative LL Moments

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Plate Centre Stress Summary

			Shear		Membrane			Bending		
	Plate	L/C	Qx (psi)	Qy (psi)	Sx (psi)	Sy (psi)	Sxy (psi)	Mx (lb'in/in)	My (lb'in/in)	Mxy (lb'in/in)
Max Qx	433	100:DC LOADS	**0.000**	-0.316	-229.253	30.320	-0.000	472.265	314.692	-0.000
Min Qx	429	100:DC LOADS	**-0.000**	2.184	-249.806	33.872	-0.000	445.580	119.359	-0.000
Max Qy	419	100:DC LOADS	0.000	**3.196**	-178.009	20.372	-0.000	350.055	-101.967	-0.000
Min Qy	437	100:DC LOADS	-0.000	**-3.200**	-177.656	20.425	-0.000	349.601	-103.771	-0.000
Max Sx	437	100:DC LOADS	-0.000	-3.200	**-177.656**	20.425	-0.000	349.601	-103.771	-0.000
Min Sx	428	100:DC LOADS	-0.000	-2.185	**-249.820**	33.866	0.000	445.562	119.197	0.000
Max Sy	429	100:DC LOADS	-0.000	2.184	-249.806	**33.872**	-0.000	445.580	119.359	-0.000
Min Sy	419	100:DC LOADS	0.000	3.196	-178.009	**20.372**	-0.000	350.055	-101.967	-0.000
Max Sxy	424	100:DC LOADS	0.000	-1.934	-239.918	32.884	**0.000**	445.409	165.494	0.000
Min Sxy	425	100:DC LOADS	0.000	2.319	-245.056	33.383	**-0.000**	438.414	100.432	-0.000
Max Mx	427	100:DC LOADS	0.000	0.066	-244.751	32.099	-0.000	**483.777**	306.548	0.000
Min Mx	437	100:DC LOADS	-0.000	-3.200	-177.656	20.425	-0.000	**349.601**	-103.771	-0.000
Max My	423	100:DC LOADS	0.000	0.314	-229.394	30.271	0.000	472.416	**315.016**	-0.000
Min My	437	100:DC LOADS	-0.000	-3.200	-177.656	20.425	-0.000	349.601	**-103.771**	-0.000
Max Mxy	427	100:DC LOADS	0.000	0.066	-244.751	32.099	-0.000	483.777	306.548	**0.000**
Min Mxy	419	100:DC LOADS	0.000	3.196	-178.009	20.372	-0.000	350.055	-101.967	**-0.000**

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	Part 6 foot spacing 70 foot span		
Job Title Emperical Deck Design	Ref		
	By GIE	Date 05-July-13	Chd
Client FDOT	File 6-70.std	Date/Time 15-Dec-2013 13:09	

Plate Centre Stress Summary

			Shear		Membrane			Bending		
	Plate	L/C	Qx (psi)	Qy (psi)	Sx (psi)	Sy (psi)	Sxy (psi)	Mx (lb·in/in)	My (lb·in/in)	Mxy (lb·in/in)
Max Qx	427	37:FUTURE W	**0.000**	-0.006	-15.116	2.083	-0.000	33.923	38.256	0.000
Min Qx	424	37:FUTURE W	**-0.000**	-0.357	-14.665	2.068	0.000	29.667	20.723	0.000
Max Qy	419	37:FUTURE W	0.000	**0.453**	-10.065	0.977	-0.000	21.677	19.213	-0.000
Min Qy	437	37:FUTURE W	-0.000	**-0.453**	-10.046	0.979	-0.000	21.644	19.128	0.000
Max Sx	437	37:FUTURE W	-0.000	-0.453	**-10.046**	0.979	-0.000	21.644	19.128	0.000
Min Sx	428	37:FUTURE W	0.000	-0.358	**-15.478**	2.212	0.000	28.247	7.001	0.000
Max Sy	429	37:FUTURE W	0.000	0.358	-15.478	**2.213**	-0.000	28.248	7.015	0.000
Min Sy	419	37:FUTURE W	0.000	0.453	-10.065	**0.977**	-0.000	21.677	19.213	-0.000
Max Sxy	424	37:FUTURE W	-0.000	-0.357	-14.665	2.068	**0.000**	29.667	20.723	0.000
Min Sxy	425	37:FUTURE W	-0.000	0.346	-15.079	2.140	**-0.000**	28.236	9.458	-0.000
Max Mx	423	37:FUTURE W	-0.000	-0.006	-13.861	1.842	0.000	**34.767**	52.375	-0.000
Min Mx	437	37:FUTURE W	-0.000	-0.453	-10.046	0.979	-0.000	**21.644**	19.128	0.000
Max My	420	37:FUTURE W	0.000	0.100	-11.139	1.225	-0.000	32.058	**63.893**	0.000
Min My	428	37:FUTURE W	0.000	-0.358	-15.478	2.212	0.000	28.247	**7.001**	0.000
Max Mxy	420	37:FUTURE W	0.000	0.100	-11.139	1.225	-0.000	32.058	63.893	**0.000**
Min Mxy	423	37:FUTURE W	-0.000	-0.006	-13.861	1.842	0.000	34.767	52.375	**-0.000**

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Appendix F: Finite element model sample, 12-foot beam spacing

FINITE ELEMENT DECK DESIGN **BEAM SPACING = 12 FEET**

Codes and Specification

- "AASHTO LRFD Bridge Design Specifications "American Association of State Highway and Transportation Officials (AASHTO), 6th Edition, 2012 with Interims through 2013.
- "Florida Department of Transportation Structures Design Guidelines for Load and Resistance Factor Design," January 2013 Edition.
- "Florida Department of Transportation Design Standards", 2014.

Reinforcing Dimensions

Input

(NOTE: For deck slabs, use same reinforcement Top & Bot. Do not include Integral Wearing Surface in "h" OR "cover")

Variables:

Beam Type:	Beam := **"FIB36"**	(Use 2, 3, 4, 5, 6, FBT72, FBT78, FIB36, FIB45, FIB54, FIB63, FIB72, FIB78)
Concrete Strength:	$f_c := \mathbf{5500}psi$	*(Assume Extremely Aggr Env., use Class IV CIP Bridge Deck Concrete, SDG Table 1.4.3-1)*
Concrete Weight:	$w_c := 150 \cdot pcf$	*(SDG Table 2.2-1)*
Aggregate Correction Factor:	$K_1 := 0.90$	*(SDG 1.4.1.A)*
Yield Strength of Reinforcing Steel:	$f_y := 60ksi$	
Width of Design Section:	$b := 1ft$	
Height of Design Section (deck thickness):	$h := \mathbf{9.0}in$	*(SDG 4.2.2.B - For new construction of "Short Bridges" other than Inverted-T Beam bridge superstructures, the minimum thickness of bridge decks cast-in-place (CIP) on beams or girders is 8-inches.*
Height of Sacrificial Wearing Surface:	$h_{sac} := 0.0in$	
Load Reduction Factor for Moment (Initial Guess):	$\phi_M := 0.90$	
Top cover:	$cover_t := \mathbf{1.5}in$	*(SDG Table 1.4.2-1)*
Bottom cover:	$cover_b := \mathbf{1.5}in$	*(SDG Table 1.4.2-1)*
Exposure Condition:	exposure_condition := **"Class 1"**	*(SDG 4.1.8)*
Beam Spacing:	$S_{beam} := \mathbf{12}ft + \mathbf{0.0}in$	
Future Wearing Surface:	$FWS := \mathbf{15}psf$	*(SDG Table 2.2-1)*
Weight of SIP Forms:	$SIP := 20psf$	
Width of Traffic Railing Barrier:	$W_{barr} := 1.5ft$	
Width of Raised Sidewalk:	$W_{sw} := 0ft$	
Traffic Railing Barrier:	$w_{barrier} := 420plf$	
Weight of Median:	$w_{med} := \mathbf{0}ft \cdot b \cdot w_c$	$w_{med} = \mathbf{0} \cdot plf$
Bridge Skew:	$Skew := mean[(\mathbf{0}),(\mathbf{0})]\ deg$	$Skew = \mathbf{0} \cdot deg$

FINITE ELEMENT DECK DESIGN **BEAM SPACING = 12 FEET**

Weight of Sidewalk: $w_{sw} := \boxed{0in + (W_{barr} + W_{sw}) \cdot 0.00} \cdot (W_{barr} + W_{sw}) \cdot w_c$ $w_{sw} = \mathbf{0} \cdot plf$

Weight of Pedestrian/Bicycle Railing and Fence: $w_{barr\_ped} := 0plf + 0plf$ $w_{barr\_ped} = \mathbf{0} \cdot plf$

Beam Properties

Constants:

Top Flange Width: $b_{tf} = \mathbf{48} \cdot in$

Web Width: $t_w = \mathbf{7} \cdot in$

Concrete Unit Weight for Modulus of Elasticity Calc: $\gamma_c := \mathbf{145}pcf$ *(SDG 1.4.1.A)*

Modulus of Elasticity Deck: $E_c := \mathbf{33000} \cdot K_1 \cdot \left(\frac{\gamma_c}{kcf}\right)^{\mathbf{1.5}} \cdot \sqrt{\frac{f_c}{ksi}} \cdot ksi$ $E_c = \mathbf{3845.8} \cdot ksi$ *(AASHTO 5.4.2.4)*

Modulus of Elasticity Reinforcing Steel: $E_s := \mathbf{29000}ksi$ *(AASHTO 5.4.3.2)*

Modular Ratio: $n := round\left(\frac{E_s}{E_c}\right)$ $n = \mathbf{8}$ *(AASHTO 5.7.1)*

Area of Deck Section: $A_c := h \cdot b$ $A_c = \mathbf{108} \cdot in^2$

Crack Control Exposure Condition Factor: $\gamma_e := \begin{cases} \mathbf{1.00} & \text{if exposure\_condition} = \text{"Class 1"} \\ \mathbf{0.75} & \text{otherwise} \end{cases}$ $\gamma_e = \mathbf{1}$ *(AASHTO 5.7.3.4)*

Design Moment

Location of Negative Live Load Design Moment:

AASHTO LRFD 4.6.2.1.6 --> The design section for negative moments and shear forces, where investigated, may be taken as follows:

For precast I-shaped concrete beams ....crossection (k) from Table 4.6.2.2.1-1:
one-third the flange width, but not exceeding 15.0 in from the centerline of support.

The negative live load design moment is taken at a distance from the supports: $Loc_{negative} := min\left(\frac{1}{3} \cdot b_{tf}, \mathbf{15} \cdot in\right)$ $Loc_{negative} = \mathbf{15.0} \cdot in$

AASHTO Table A4-1 - Deck Slab Design Table

Dead Load Moments for Moment Analysis:

"DC" loads include the dead load of structural components and non-structural attachments

Self-weight of Deck Slab: $w_{slab} := \left[(h + h_{sac}) \cdot b\right] \cdot w_c$ $w_{slab} = \mathbf{0.113} \cdot klf$

Weight of Traffic Railing Barriers: $P_{barrier} := w_{barrier} \cdot b$ $P_{barrier} = \mathbf{0.420} \cdot kip$

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FINITE ELEMENT DECK DESIGN **BEAM SPACING = 12 FEET**

Weight of Pedestrian/Bicycle Railing and Fence:	$P_{barTypeK} := w_{barr\_ped} \cdot b$	$P_{barTypeK} = \mathbf{0.000} \cdot kip$
Weight of Median:	$w_{median} := w_{med}$	$w_{median} = \mathbf{0.000} \cdot klf$
Weight of Sidewalk:	$w_{sw} := \dfrac{w_{sw} \cdot b}{W_{sw} + W_{barr}}$	$w_{sw} = \mathbf{0.000} \cdot klf$
Stay-in-Place Forms:	$w_{sip} := SIP \cdot b$	$w_{sip} = \mathbf{0.020} \cdot klf$

"DW" loads include the dead load of a future wearing surface and utilities

Weight of Future Wearing Surface:	$w_{fws} := FWS \cdot b$	$w_{fws} = \mathbf{0.015} \cdot klf$

Max. Positive Live Load Moment:

From Finite Element Model: $M_{LL\_pos} := \mathbf{4.080} \cdot kip \cdot \dfrac{ft}{ft}$ $M_{LL\_pos} = \mathbf{4.08} \cdot kip \cdot \dfrac{ft}{ft}$

Max. Negative Live Load Moment:

From Finite Element Model:

From Finite Element Models $M_{LL\_neg} := \mathbf{2.140} \cdot kip \cdot \dfrac{ft}{ft}$

$M_{LL\_neg} = \mathbf{2.14} \cdot kip \cdot \dfrac{ft}{ft}$

FINITE ELEMENT DECK DESIGN **BEAM SPACING = 12 FEET**

Summary of Moments:

Max Moments between the beams:

Max. Positive Service DC Moment: $M_{DC\_Pos} := 1.48 \cdot kip \cdot \frac{ft}{ft}$ $M_{DC\_Pos} = 1.48 \cdot kip \cdot \frac{ft}{ft}$ (See STAAD Output)

Max. Negative Service DC Moment: $M_{DC\_Neg} := 0.159 \cdot kip \cdot \frac{ft}{ft}$ $M_{DC\_Neg} = 0.159 \cdot kip \cdot \frac{ft}{ft}$ (See STAAD Output)

Max. Positive Service DW Moment: $M_{DW\_Pos} := 0.179 \cdot kip \cdot \frac{ft}{ft}$ $M_{DW\_Pos} = 0.179 \cdot kip \cdot \frac{ft}{ft}$ (See STAAD Output)

Max. Negative Service DW Moment $M_{DW\_Neg} := 0.029 \cdot kip \cdot \frac{ft}{ft}$ $M_{DW\_Neg} = 0.029 \cdot kip \cdot \frac{ft}{ft}$ (See STAAD Output)

Load Combinations:

Maximum Service I Moment: $M_{Service} := \max(M_{DC\_Pos} + M_{DW\_Pos} + M_{LL\_pos}, M_{DC\_Neg} + M_{DW\_Neg} + M_{LL\_neg}) \cdot ft$

$M_{Service} = 5.739 \cdot kip \cdot ft$

Maximum Strength I Moment: $M_{Strength} := \max\begin{pmatrix} 1.25M_{DC\_Pos} + 1.50 \cdot M_{DW\_Pos} \cdots, 1.25M_{DC\_Neg} + 1.5 \cdot M_{DW\_Neg} \cdots \\ +1.75M_{LL\_pos} \qquad +1.75M_{LL\_neg} \end{pmatrix} \cdot ft$

$M_{Strength} = 9.258 \cdot kip \cdot ft$

Applied Moment: $M_{applied} := M_{Strength}$ $M_{applied} = 9.258 \cdot kip \cdot ft$

Flexure Reinforcement

Minimum Reinforcement: *(AASHTO 5.7.3.3.2 & See AASHTO 5.7.2 for Design Assumptions)*

Modulus of Rapture: *(AASHTO 5.4.2.6 & SDG 1.4.1.B)* $f_r := 0.24 \cdot \sqrt{\frac{f_c}{ksi}} \cdot ksi$ $f_r = 562.85 \cdot psi$

Moment of Inertia of Slab Section: $I_g := \frac{b \cdot h^3}{12}$ $I_g = 729 \cdot in^4$

Distance from the Extreme Tensile Fiber to the Neutral Axis of the Composite Section: $y_t := \frac{h}{2}$ $y_t = 4.5 \cdot in$

Cracking Moment: $M_{cr} := \frac{f_r \cdot I_g}{y_t}$ $M_{cr} = 7.598 \cdot kip \cdot ft$

FINITE ELEMENT DECK DESIGN **BEAM SPACING = 12 FEET**

Cracking Moment Limit: $1.2 \cdot M_{cr} = \mathbf{9.118} \cdot \text{kip} \cdot \text{ft}$

Des gin Ultimate Moment:
$$M_u := \begin{vmatrix} M_{Strength} \ \text{if} \ M_{Strength} \geq \mathbf{1.2} M_{cr} \\ \min(\mathbf{1.33} M_{Strength}, \mathbf{1.2} M_{cr}) \ \text{otherwise} \end{vmatrix} \qquad M_u = \mathbf{9.258} \cdot \text{kip} \cdot \text{ft}$$

Distance from Extreme Compressive Fiber to Centroid of Reinforcing Steel:
$$d_e := h - cover_b - \frac{d_{bar}(bar_{main})}{\mathbf{2}} \qquad d_e = \mathbf{7.188} \cdot \text{in}$$

Nominal Strength Coefficient of Resistance:
$$R_u := \frac{M_u}{\phi_M \cdot b \cdot d_e^{\mathbf{2}}} \qquad R_u = \mathbf{199.133} \cdot \text{psi}$$

$$m := \frac{f_y}{\mathbf{0.85} \cdot f_c} \qquad m = \mathbf{12.834}$$

ACI ρ Equation:
$$\rho := \frac{\mathbf{1}}{m} \cdot \left(\mathbf{1} - \sqrt{\mathbf{1} - \frac{\mathbf{2} \cdot \frac{R_u}{\text{psi}} \cdot m}{\frac{f_y}{\text{psi}}}} \right) \qquad \rho = \mathbf{0.003393}$$

$$A_{sreqdpos} := \rho \cdot b \cdot d_e \qquad A_{sreqdpos} = \mathbf{0.293} \cdot \text{in}^{\mathbf{2}}$$

Minimum Required A_s between beams
$$A_{sMinReq} := A_{sreqdpos} \qquad A_{sMinReq} = \mathbf{0.293} \cdot \text{in}^{\mathbf{2}}$$

Use Main Reinforcing: $bar_{main} \equiv 5$ $\quad Spa_{main} := \mathbf{12.00} \text{in}$

Diameter of Bar:
$$d_b := d_{bar}(bar_{main}) \qquad d_b = \mathbf{0.625} \cdot \text{in}$$

Area of Reinforcing in a Section 1 ft Wide:
$$A\_s(z, cc) := A_{bar}(z) \cdot \frac{\mathbf{12} \cdot \text{in}}{cc}$$

Area of Reinforcing:
$$A_{sMain} := A\_s(bar_{main}, Spa_{main}) \qquad A_{sMain} = \mathbf{0.31} \cdot \text{in}^{\mathbf{2}}$$

Depth of Equivalent Rectangular Stress Block:
$$a := \frac{A_{sMain} \cdot f_y}{\mathbf{0.85} \cdot f_c \cdot b} \qquad a = \mathbf{0.332} \cdot \text{in}$$

Ratio of Reinforcement Provided:
$$\rho := \frac{A_{sMain}}{b \cdot d_e} \qquad \rho = \mathbf{0.0036}$$

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FINITE ELEMENT DECK DESIGN **BEAM SPACING = 12 FEET**

Determine ϕ (Tension or Compression Controlled Section): *(AASHTO 5.7.2.1 & AASHTO 5.7.2.2)*

Determine location of N.A using Whitney Stress Block.:

$$\beta_1 := \begin{cases} \max\left[\left[0.85 - \mathbf{0.05}\cdot\left(\frac{f_c - \mathbf{4}\cdot \text{ksi}}{\text{ksi}}\right)\right], \mathbf{0.65}\right] & \text{if } f_c > \mathbf{4}\text{ksi} \\ \mathbf{0.85} & \text{otherwise} \end{cases} \qquad \beta_1 = \mathbf{0.775}$$

Distance from the Extreme Compression Fiber to the N.A.:

$$c_{comp} := \frac{a}{\beta_1} \qquad c_{comp} = \mathbf{0.428}\cdot\text{in}$$

Actual Tensile Strain in Extreme Tension Steel:

$$\varepsilon_T := \mathbf{0.003}\cdot\frac{d_e - c_{comp}}{c_{comp}} \quad \textit{(AASHTO Figure C5.7.2.1-1)} \qquad \varepsilon_T = \mathbf{0.047}$$

Comp. and Tension Controlled Section Limits of Net Tensile Strain in the Extreme Tension Steel:

$$\varepsilon_{T\_Limits} := \begin{pmatrix} 0.002 \\ 0.005 \end{pmatrix}$$

Compression Controlled if ε_T <or= 0.0002
Tension Controlled if ε_T >or= 0.0005

Comp. and Tension Controlled Reinforced Concrete Section Resistance Factors:

$$\phi := \begin{pmatrix} 0.75 \\ \mathbf{0.90} \end{pmatrix}$$

Compression Controlled
Tension Controlled *(AASHTO 5.5.4.2.1)*

Determine Controlling Force:

$$\text{Controlling} := \begin{cases} \textbf{"Compression"} & \text{if } \varepsilon_T \le \varepsilon_{T\_Limits_0} \\ \textbf{"Tension"} & \text{if } \varepsilon_T \ge \varepsilon_{T\_Limits_1} \\ \textbf{"In Transition"} & \text{otherwise} \end{cases} \qquad \text{Controlling} = \textbf{"Tension"}$$

Determine Controlling Resistance Factor:

$$\phi_M := \begin{cases} \phi_0 & \text{if Controlling} = \textbf{"Compression"} \\ \phi_1 & \text{if Controlling} = \textbf{"Tension"} \\ \text{linterp}(\varepsilon_{T\_Limits}, \phi, \varepsilon_T) & \text{otherwise} \end{cases} \qquad \phi_M = \mathbf{0.9}$$

Factored Flexural Resistance:

$$\phi M_n := \phi_M \cdot A_{sMain} \cdot f_y \cdot \left(d_e - \frac{a}{\mathbf{2}}\right) \qquad \phi M_n = \mathbf{9.795}\cdot\text{kip}\cdot\text{ft}$$

Ultimate Moment:

$$M_u = \mathbf{9.258}\cdot\text{kip}\cdot\text{ft}$$

Check Moment Capacity:

$$\text{CheckMoment} := \begin{cases} \textbf{"OK"} & \text{if } \phi M_n \ge M_u \\ \textbf{"No Good"} & \text{if } \phi M_n < M_u \end{cases} \qquad \boxed{\text{CheckMoment} = \textbf{"OK"}}$$

FINITE ELEMENT DECK DESIGN **BEAM SPACING = 12 FEET**

Crack Control Check *(AASHTO 5.7.3.4)*

Thickness of Concrete Cover Measured from Extreme Tension Fiber to Center of the Flexural Reinforcement Located Closest Thereto: $d_c := cover_b + \frac{d_b}{2}$ $d_c = 1.812 \cdot in$

Depth of Neutral Axis $f(x) := \frac{b \cdot x^2}{2 \cdot A_{sMain} \cdot n} + x - d_e$

$x := root(f(x), x, 0, d_e)$ $x = 1.529 \cdot in$

Tensile Stress in Reinforcement at the Service Limit State: $f_{ss} := \frac{M_{Service}}{A_{sMain} \cdot \left(d_e - \frac{x}{3}\right)}$ $f_{ss} = 33.268 \cdot ksi$

Ratio of Flexural Strain at the Extreme Tension Face to the Strain at the Centroid of the Reinforcement Layer Nearest the Tension Face: $\beta_s := 1 + \frac{d_c}{0.7 \cdot (h - d_c)}$ *(AASHTO 5.7.3.4-1)* $\beta_s = 1.36$

Maximum Reinforcement Spacing for Crack Control: $s_{max} := \left(\frac{700 \cdot \gamma_e}{\beta_s \cdot \frac{f_{ss}}{ksi}} - 2 \cdot \frac{d_c}{in}\right) \cdot in$ *(AASHTO 5.7.3.4-1)* $s_{max} = 11.844 \cdot in$

Reinforcement Spacing Provided: $s_{actual} := Spa_{main}$ $s_{actual} = 12 \cdot in$

Check Spacing: CheckSpacing := | **"No Good"** if $s_{actual} > s_{max}$ / **"OK"** otherwise — CheckSpacing = **"No Good"**

Distribution Reinforcement *(AASHTO 9.7.3.2)*

For primary reinforcement perpendicular to traffic

Distance Between Beam Flange Tips: $D1 := S_{beam} - b_{tf}$ $D1 = 8 \, ft$

Flange Overhang: $D2 := b_{tf} - t_w$ $D2 = 3.417 \, ft$

Effective Span Length: $sp_{eff} := D1 + D2$ *(AASHTO 9.7.2.3)* $sp_{eff} = 11.417 \, ft$

Distribution Reinforcement %: $DR := \min\left(\frac{220}{\sqrt{\frac{sp_{eff}}{ft}}}, 67\right) \cdot \%$ $DR = 65.111 \cdot \%$

FINITE ELEMENT DECK DESIGN **BEAM SPACING = 12 FEET**

Area of Steel Required: $A_{sDR} := DR \cdot A_{sMain}$ $A_{sDR} = 0.202 \cdot in^2$

Use Dist. Reinforcement: $bar_{DP} := 5$ $Spa_{DP} := 12.0in$ (Max.)

Area of Reinforcing Provided: $A_{sDP} := A\_s(bar_{DP}, Spa_{DP})$ $A_{sDP} = 0.31 \cdot in^2$

$$CheckDistRein := \begin{cases} \text{"OK"} & \text{if } A_{sDP} \ge A_{sDR} \\ \text{"No Good"} & \text{otherwise} \end{cases}$$

$CheckDistRein = \text{"OK"}$

Temperature & Shrinkage Reinforcement:

(AASHTO 5.10.8.2)

Area of Steel Required for Temp & Shrinkage:

$$A_{s\_ts} := \begin{cases} 0.11 \dfrac{in^2}{ft} & \text{if } \dfrac{1.30 \cdot \dfrac{kip}{in \cdot ft} \cdot b \cdot h}{2 \cdot (b+h) \cdot f_y} \le 0.11 \dfrac{in^2}{ft} \\ 0.60 \dfrac{in^2}{ft} & \text{if } \dfrac{1.30 \cdot \dfrac{kip}{in \cdot ft} \cdot b \cdot h}{2 \cdot (b+h) \cdot f_y} \ge 0.60 \dfrac{in^2}{ft} \\ \dfrac{1.30 \cdot \dfrac{kip}{in \cdot ft} \cdot b \cdot h}{2 \cdot (b+h) \cdot f_y} & \text{otherwise} \end{cases}$$

$A_{s\_ts} = 0.11 \cdot \dfrac{in^2}{ft}$

Use: $bar_{TS} := 5$ *(SDG 4.2.11 - #4 Min.)* $Spa_{TS} := 12.0in$ *(SDG 4.2.11 - 12in Max.)*

Area of Temp and Shrink Reinforcing: $A_{sTS} := A\_s(bar_{TS}, Spa_{TS})$ $A_{sTS} = 0.31 \cdot in^2$

$$Check_{TS} := \begin{cases} \text{"OK"} & \text{if } A_{sDR} \ge A_{s\_ts} \cdot ft \wedge A_{sMain} \ge A_{s\_ts} \cdot ft \wedge A_{sTS} \ge A_{s\_ts} \cdot ft \\ \text{"No Good"} & \text{otherwise} \end{cases}$$

$Check_{TS} = \text{"OK"}$

Maximum Spacing of Temperature and Shrinkage Reinforcement: $s_{TSmax} := \min(3 \cdot h, 18in, 12in)$ $s_{TSmax} = 12 \cdot in$

$$Check_{Spa} := \begin{cases} \text{"OK"} & \text{if } Spa_{DP} \le s_{TSmax} \wedge Spa_{main} \le s_{TSmax} \wedge Spa_{TS} \le s_{TSmax} \\ \text{"No Good"} & \text{otherwise} \end{cases}$$

$Check_{Spa} = \text{"OK"}$

FINITE ELEMENT DECK DESIGN **BEAM SPACING = 12 FEET**

Deck Reinforcement Summary

Main Reinforcement, Transverse (Bot.):	**Use #**	$bar_{main} = \mathbf{5}$	**bars at**	$Spa_{main} = \mathbf{12} \cdot in$	
Main Reinforcement, Transverse (Top):	**Use #**	$bar_{main} = \mathbf{5}$	**bars at**	$Spa_{main} = \mathbf{12} \cdot in$	
Distribution Reinforcement, Longitudinal (Bottom):	**Use #**	$bar_{DP} = \mathbf{5}$	**bars at**	$Spa_{DP} = \mathbf{12} \cdot in$	(Max.)
Temperature and Shrinkage, Longitudinal (Top):	**Use #**	$bar_{TS} = \mathbf{5}$	**bars at**	$Spa_{TS} = \mathbf{12} \cdot in$	(Max.)

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	Part 12 foot spacing 70 foot span		
Job Title Emperical Deck Design	Ref		
	By GIE	Date 13-Oct-13	Chd
Client FDOT	File 12-70.std	Date/Time 13-Oct-2013 16:51	

Job Information

	Engineer	Checked	Approved
Name:	GIE		
Date:	13-Oct-13		

Comments

FIB 36

Structure Type	SPACE FRAME

Number of Nodes	836	Highest Node	836
Number of Elements	228	Highest Beam	1009
Number of Plates	770	Highest Plate	805

Number of Basic Load Cases	3
Number of Combination Load Cases	1

Included in this printout are data for:

All	The Whole Structure

Included in this printout are results for load cases:

Type	L/C	Name
Generation	2	LOAD GENERATION, LOAD #2, (1 of 21)
Generation	3	LOAD GENERATION, LOAD #3, (2 of 21)
Generation	4	LOAD GENERATION, LOAD #4, (3 of 21)
Generation	5	LOAD GENERATION, LOAD #5, (4 of 21)
Generation	6	LOAD GENERATION, LOAD #6, (5 of 21)
Generation	7	LOAD GENERATION, LOAD #7, (6 of 21)
Generation	8	LOAD GENERATION, LOAD #8, (7 of 21)
Generation	9	LOAD GENERATION, LOAD #9, (8 of 21)
Generation	10	LOAD GENERATION, LOAD #10, (9 of 21)
Generation	11	LOAD GENERATION, LOAD #11, (10 of 2
Generation	12	LOAD GENERATION, LOAD #12, (11 of 2
Generation	13	LOAD GENERATION, LOAD #13, (12 of 2
Generation	14	LOAD GENERATION, LOAD #14, (13 of 2
Generation	15	LOAD GENERATION, LOAD #15, (14 of 2
Generation	16	LOAD GENERATION, LOAD #16, (15 of 2
Generation	17	LOAD GENERATION, LOAD #17, (16 of 2
Generation	18	LOAD GENERATION, LOAD #18, (17 of 2
Generation	19	LOAD GENERATION, LOAD #19, (18 of 2
Generation	20	LOAD GENERATION, LOAD #20, (19 of 2
Generation	21	LOAD GENERATION, LOAD #21, (20 of 2
Generation	22	LOAD GENERATION, LOAD #22, (21 of 2
Generation	23	LOAD GENERATION, LOAD #23, (1 of 13)
Generation	24	LOAD GENERATION, LOAD #24, (2 of 13)
Generation	25	LOAD GENERATION, LOAD #25, (3 of 13)

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Job Information Cont...

Type	L/C	Name
Generation	26	LOAD GENERATION, LOAD #26, (4 of 13)
Generation	27	LOAD GENERATION, LOAD #27, (5 of 13)
Generation	28	LOAD GENERATION, LOAD #28, (6 of 13)
Generation	29	LOAD GENERATION, LOAD #29, (7 of 13)
Generation	30	LOAD GENERATION, LOAD #30, (8 of 13)
Generation	31	LOAD GENERATION, LOAD #31, (9 of 13)
Generation	32	LOAD GENERATION, LOAD #32, (10 of 1:
Generation	33	LOAD GENERATION, LOAD #33, (11 of 1:
Generation	34	LOAD GENERATION, LOAD #34, (12 of 1:
Generation	35	LOAD GENERATION, LOAD #35, (13 of 1:

Section Properties

Prop	Section	Area (in^2)	I_{yy} (in^4)	I_{zz} (in^4)	J (in^4)	Material
3	Rect 36.00x36.00	1.3E+3	140E+3	140E+3	236E+3	CONCRETE
4	Rect 18.00x18.00	324.000	8.75E+3	8.75E+3	14.8E+3	CONCRETE
5	FIB-36	810.187	81.4E+3	128E+3	31.1E+3	CONCRETE
6	TYPEFBARRIERL	401.375	6.5E+3	36E+3	14.1E+3	CONCRETE
7	TYPEFBARRIER	403.781	6.65E+3	36.1E+3	14.4E+3	CONCRETE

Plate Thickness

Prop	Node A (in)	Node B (in)	Node C (in)	Node D (in)	Material
1	8.000	8.000	8.000	8.000	CONCRETE
2	12.000	12.000	12.000	12.000	CONCRETE

Materials

Mat	Name	E (kip/in^2)	ν	Density (kip/in^3)	α (/°F)
1	STEEL	29E+3	0.300	0.000	6E -6
2	STAINLESSSTEEL	28E+3	0.300	0.000	10E -6
3	ALUMINUM	10E+3	0.330	0.000	13E -6
4	CONCRETE	3.15E+3	0.170	0.000	5E -6

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Supports

Node	X (kip/in)	Y (kip/in)	Z (kip/in)	rX (kip˙ft/deg)	rY (kip˙ft/deg)	rZ (kip˙ft/deg)
1	Fixed	Fixed	Fixed	Fixed	Fixed	Fixed
2	Fixed	Fixed	Fixed	Fixed	Fixed	Fixed
3	Fixed	Fixed	Fixed	Fixed	Fixed	Fixed
4	Fixed	Fixed	Fixed	Fixed	Fixed	Fixed
810	Fixed	Fixed	Fixed	Fixed	Fixed	Fixed
811	Fixed	Fixed	Fixed	Fixed	Fixed	Fixed
812	Fixed	Fixed	Fixed	Fixed	Fixed	Fixed
813	Fixed	Fixed	Fixed	Fixed	Fixed	Fixed

Basic Load Cases

Number	Name
1	SELFWEIGHT
36	SIP
37	FWS

Moving Load Definition : Type 1

Type	vs (ft)	Factor
HS20	-	1.330

Moving Load Definition : Type 2

Width (ft)
14.000

Force (kip)	Distance (ft)
42.560	-
42.560	6.000

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Beam Displacement Detail Summary

Displacements shown in italic indicate the presence of an offset

	Beam	L/C	d (ft)	X (in)	Y (in)	Z (in)	Resultant (in)
Max X	30	7:LOAD GENE	2.000	**0.011**	-0.036	0.000	0.037
Min X	955	27:LOAD GENI	1.800	**-0.010**	-0.141	0.001	0.141
Max Y	893	33:LOAD GENI	1.000	-0.000	**0.005**	-0.004	0.007
Min Y	895	27:LOAD GENI	1.000	-0.000	**-0.309**	0.000	0.309
Max Z	892	24:LOAD GENI	1.400	-0.000	0.005	**0.004**	0.007
Min Z	893	33:LOAD GENI	0.400	-0.000	0.005	**-0.004**	0.007
Max Rst	895	27:LOAD GENI	1.000	-0.000	-0.309	0.000	**0.309**

Plate Centre Stress Summary

			Shear		Membrane			Bending		
	Plate	L/C	Qx (psi)	Qy (psi)	Sx (psi)	Sy (psi)	Sxy (psi)	Mx (lb·in/in)	My (lb·in/in)	Mxy (lb·in/in)
Max Qx	307	27:LOAD GENI	**12.186**	4.950	-142.422	27.098	71.310	1.21E+3	3.47E+3	-56.688
Min Qx	527	27:LOAD GENI	**-12.186**	4.950	-142.417	27.097	-71.311	1.21E+3	3.47E+3	56.692
Max Qy	489	33:LOAD GENI	0.976	**9.316**	-183.803	32.994	-27.182	1.6E+3	3.97E+3	67.490
Min Qy	496	35:LOAD GENI	1.766	**-12.076**	-136.366	19.367	32.274	1.05E+3	2.08E+3	-112.337
Max Sx	44	27:LOAD GENI	6.218	-0.061	**223.140**	0.012	127.151	-664.359	7.193	-492.752
Min Sx	484	27:LOAD GENI	1.018	-7.307	**-190.990**	34.456	23.708	1.6E+3	3.76E+3	61.617
Max Sy	37	23:LOAD GENI	-2.952	-3.572	30.307	**131.190**	53.773	-317.706	-466.871	-172.776
Min Sy	477	23:LOAD GENI	8.999	-1.093	15.093	**-123.129**	26.502	1.34E+3	779.121	-164.737
Max Sxy	44	27:LOAD GENI	6.218	-0.061	223.140	0.012	**127.151**	-664.359	7.193	-492.752
Min Sxy	792	27:LOAD GENI	-6.218	-0.061	223.139	0.013	**-127.150**	-664.354	7.193	492.748
Max Mx	483	27:LOAD GENI	0.919	8.860	-187.633	33.858	-29.814	**1.61E+3**	4.01E+3	75.513
Min Mx	805	35:LOAD GENI	1.193	0.775	-79.984	0.934	-22.038	**-896.265**	-89.302	1.195
Max My	424	30:LOAD GENI	0.000	-6.307	-160.723	33.163	-0.000	1.43E+3	**4.08E+3**	-0.003
Min My	423	24:LOAD GENI	-0.000	-6.879	-91.153	8.579	-0.000	-65.929	**-2.14E+3**	-0.003
Max Mxy	787	27:LOAD GENI	0.565	-2.094	0.131	-16.241	-8.977	137.753	-100.858	**877.713**
Min Mxy	39	27:LOAD GENI	-0.565	-2.094	0.131	-16.241	8.977	137.756	-100.861	**-877.722**

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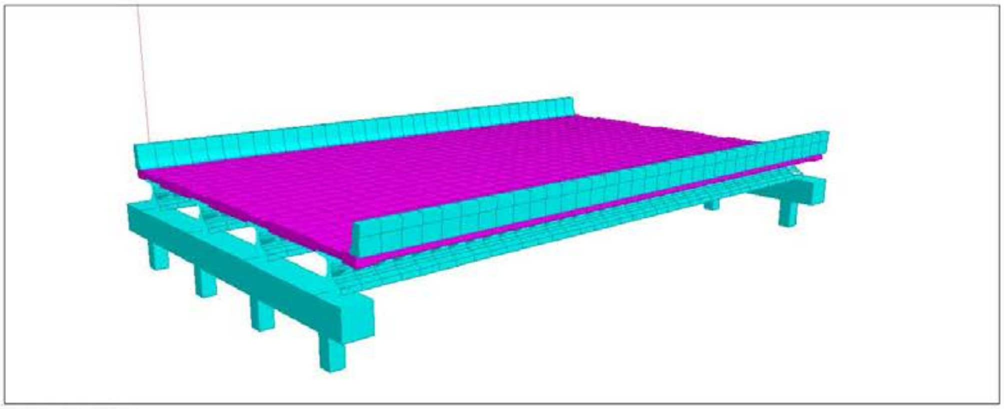
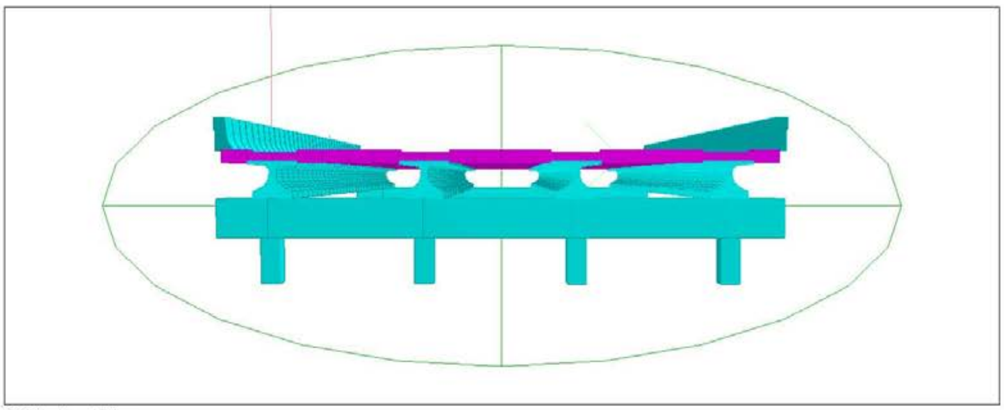


3D Rendered View



3D Rendered View

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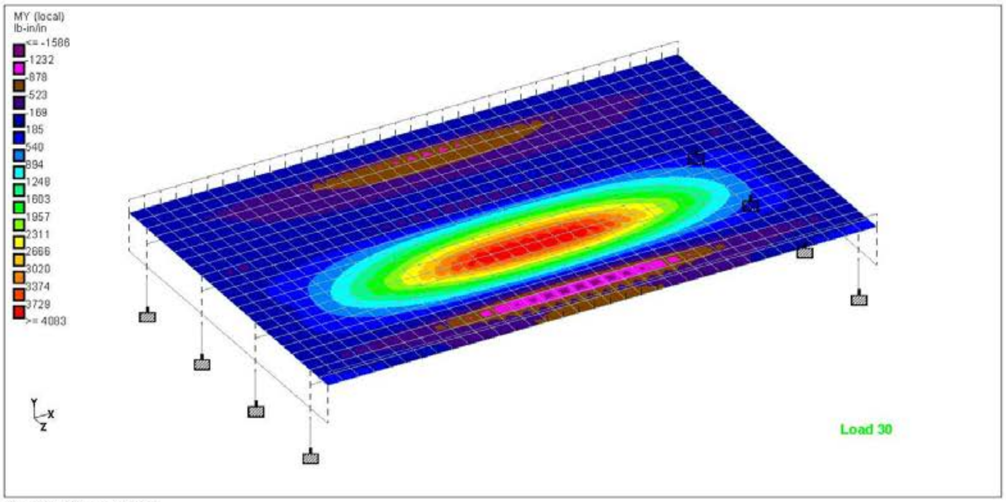
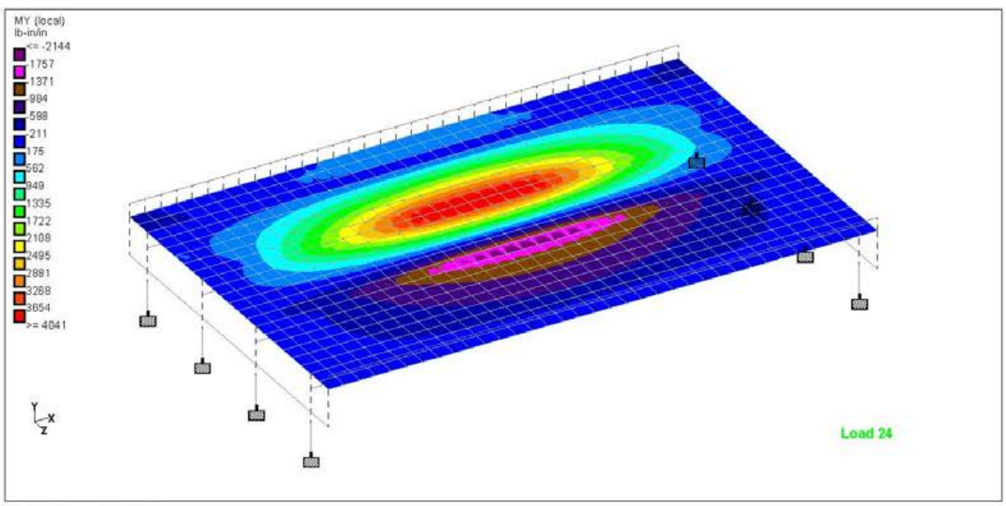


Positive Moment MY



Negative Moment MY

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Plate Centre Stress Summary

			Shear		Membrane			Bending		
	Plate	L/C	Qx (psi)	Qy (psi)	Sx (psi)	Sy (psi)	Sxy (psi)	Mx (lb'in/in)	My (lb'in/in)	Mxy (lb'in/in)
Max Qx	429	100:DC LOADS	**0.000**	-6.054	-147.073	15.500	0.000	900.234	-158.716	-0.000
Min Qx	412	100:DC LOADS	**-0.000**	6.051	-147.427	15.456	-0.000	901.117	-155.546	-0.000
Max Qy	412	100:DC LOADS	-0.000	**6.051**	-147.427	15.456	-0.000	901.117	-155.546	-0.000
Min Qy	429	100:DC LOADS	0.000	**-6.054**	-147.073	15.500	0.000	900.234	-158.716	-0.000
Max Sx	429	100:DC LOADS	0.000	-6.054	**-147.073**	15.500	0.000	900.234	-158.716	-0.000
Min Sx	418	100:DC LOADS	0.000	5.613	**-202.522**	27.700	0.000	1.13E+3	-109.480	-0.000
Max Sy	423	100:DC LOADS	0.000	-5.614	-202.445	**27.724**	0.000	1.13E+3	-109.989	0.000
Min Sy	413	100:DC LOADS	0.000	3.936	-151.595	**14.486**	-0.000	1.15E+3	890.077	0.000
Max Sxy	423	100:DC LOADS	0.000	-5.614	-202.445	27.724	**0.000**	1.13E+3	-109.989	0.000
Min Sxy	424	100:DC LOADS	0.000	5.235	-197.866	27.153	**-0.000**	1.15E+3	71.215	-0.000
Max Mx	420	100:DC LOADS	-0.000	1.171	-190.736	20.982	0.000	**1.42E+3**	1.39E+3	-0.000
Min Mx	429	100:DC LOADS	0.000	-6.054	-147.073	15.500	0.000	**900.234**	-158.716	-0.000
Max My	415	100:DC LOADS	0.000	-0.804	-168.308	17.161	-0.000	1.34E+3	**1.48E+3**	0.000
Min My	429	100:DC LOADS	0.000	-6.054	-147.073	15.500	0.000	900.234	**-158.716**	-0.000
Max Mxy	415	100:DC LOADS	0.000	-0.804	-168.308	17.161	-0.000	1.34E+3	1.48E+3	**0.000**
Min Mxy	421	100:DC LOADS	0.000	-1.172	-190.721	20.987	0.000	1.42E+3	1.39E+3	**-0.000**

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Plate Centre Stress Summary

			Shear		Membrane			Bending		
	Plate	L/C	Qx (psi)	Qy (psi)	Sx (psi)	Sy (psi)	Sxy (psi)	Mx (lb'in/in)	My (lb'in/in)	Mxy (lb'in/in)
Max Qx	424	37:FWS	**0.000**	0.739	-15.673	2.375	-0.000	61.681	1.535	0.000
Min Qx	419	37:FWS	**-0.000**	0.457	-15.552	2.105	-0.000	80.912	87.828	-0.000
Max Qy	412	37:FWS	-0.000	**0.778**	-10.374	0.915	-0.000	39.345	6.793	-0.000
Min Qy	429	37:FWS	0.000	**-0.779**	-10.351	0.917	-0.000	39.270	6.572	-0.000
Max Sx	429	37:FWS	0.000	-0.779	**-10.351**	0.917	-0.000	39.270	6.572	-0.000
Min Sx	418	37:FWS	-0.000	0.755	**-16.168**	2.462	-0.000	58.674	-28.593	-0.000
Max Sy	423	37:FWS	-0.000	-0.755	-16.162	**2.463**	0.000	58.666	-28.579	0.000
Min Sy	412	37:FWS	-0.000	0.778	-10.374	**0.915**	-0.000	39.345	6.793	-0.000
Max Sxy	423	37:FWS	-0.000	-0.755	-16.162	2.463	**0.000**	58.666	-28.579	0.000
Min Sxy	412	37:FWS	-0.000	0.778	-10.374	0.915	**-0.000**	39.345	6.793	-0.000
Max Mx	420	37:FWS	-0.000	0.152	-15.242	1.925	-0.000	**91.104**	145.525	-0.000
Min Mx	429	37:FWS	0.000	-0.779	-10.351	0.917	-0.000	**39.270**	6.572	-0.000
Max My	414	37:FWS	0.000	0.167	-11.797	1.101	-0.000	80.734	**178.960**	0.000
Min My	418	37:FWS	-0.000	0.755	-16.168	2.462	-0.000	58.674	**-28.593**	-0.000
Max Mxy	422	37:FWS	-0.000	-0.457	-15.549	2.106	0.000	80.907	87.836	**0.000**
Min Mxy	419	37:FWS	-0.000	0.457	-15.552	2.105	-0.000	80.912	87.828	**-0.000**

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Vita

Georges El-Gharib was born . He relocated to Cleveland, Ohio in 1993. He earned his Bachelor in Civil Engineering degree in May 2006 from Cleveland State University and graduated with honors. After working for two consulting engineering companies as a bridge engineer and structural engineer in Cleveland, Ohio, he moved to Jacksonville, Florida in 2009. He started his graduate studies in civil engineering at the University of North Florida in August 2012, and is expected to obtain his Masters of Science in Civil Engineering degree in April 2014.

Mr. El-Gharib is a registered Professional Engineer with the State of Florida Board of Professional Engineers, and is a member of the Tau Beta Pi Engineering honor society.

Mr. El-Gharib continues to work in the bridge engineering field and his most recent position as a project engineer he conducted several bridge load rating analysis that included prestressed concrete beams, steel plate girders and steel box girders. He also was responsible for several new bridge designs and existing bridge rehabilitation projects for FDOT and other states.